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INTRODUCTION

Purpose

This second progress report presents performance characteristics of possible anchors for the Ocean Thermal Energy Conversion power plant (OTEC). The first report (Valent et al, 1976) gave a general description of possible OTEC anchor concepts.

Procedure

Each anchor type presented in the first OTEC anchor report was evaluated using available data and standard analysis techniques. All calculations were made assuming that load was statically applied. The advantages and limitations of combining several anchors of the same type to achieve desired capacity were discussed.

Anchor holding capacities for OTEC were calculated for the soil categories described in the first report. A summary of these soil categories and assumed anchor loading is given in the next section.

ENVIRONMENT

Site Categories

The sediment strength profiles described in the first report were grouped into three site categories. One shear strength profile was developed to represent each category (Figure 1). These three site categories, plus two additional categories for sands and for outcropping and near-surface seafloor rock respectively, were used as representative OTEC seafloor environments. The selection, sizing, and comparing of anchor types and combinations refers back to these baseline categories.

Table 1 is a summary listing of site category characteristics. Note that each category has been assigned a descriptive word title in Table 1; for example, the sediments of site category A can be descriptively referred to as clays and clayey silts.

For site category D, sands, a predominantly quartz grain material is assumed having a friction angle of 0.52 rad (30 deg) and a submerged unit weight of 6.8kN/m^3 (43 pcf). Such material under repeated loading may be susceptible to liquefaction. If the median grain size (D_{50}) is found to lie between 0.02 and 0.2 mm, then the mooring system should be designed to reduce the effects of repeated loading or high factors of safety (greater than 10) should be used.

The material characteristics of site category E, that for exposed or shallowly buried rock, are not resolved at this point. The quantified

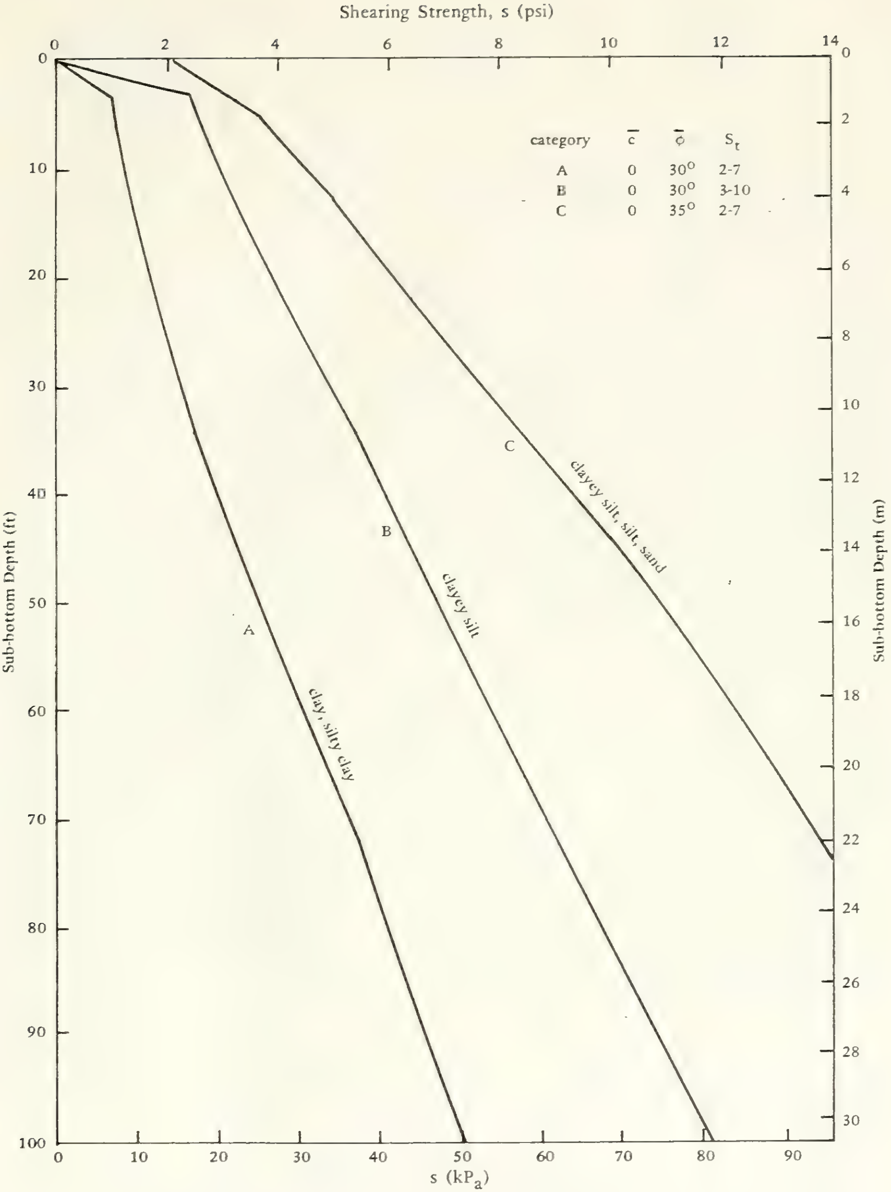


Figure 1. Properties assumed for 3 of 5 assumed seafloor categories.

Table 1. Measured/Calculated Profile Groupings and Assumed Site Category Characteristics

Category	Profile[a]	Figure[a]	Description	s(kPa/psi)	$\overline{\phi}$(rad/deg)	S_t	γ(kN/m^3/pcf)	Comments
A			CLAY; SILTY CLAY[b]	0	.52/30	2-7	4.08/26	<25% forams
	1	2	Pelagic clay, abyssal hill province	3.45/0.5	.61/35	5-7[c]		
	2	3	Clay and nannofossil calcareous ooze	0	.66/37	5		
	3	5	Soft basin material	1.38/0.2	.66/37	2-4		
	4	4	Distal turbidite (weak)	2.76/0.4	.52/30	3-5		
B			CLAYEY SILT[b]	0	.52/30	3-10	5.81/37	>60% forams
	1	3	Loose foram ooze, calcareous	0	.58/33	10		
	2	4	Distal turbidite (strong)	2.76/0.4	.52/30	3-5		
C			CLAYEY SILT, SILT[b]	0	.61/35	2-7	6.75/43	>50% forams
	1	4	Proximal turbidite (silt)	1.72/0.25	.68/39	2-3		
	2	3	Dense foram ooze, calcareous	0	.58/33	5-7		
D			SAND[b]	0	.52/30	10	6.75/43	
	1	4	Proximal turbidite (sand)	0	.52/30	potentially liquefiable		
E			SEAFLOOR ROCK, exposed or shallowly buried [b]					

[a] For data source refer to Valent et al., 1976.

[b] Values are based on engineering judgments

[c] For upper 4.6m; 3 to 5 for below 4.6 m.

rock properties are not nearly as significant in comparing anchor types as are the mechanims for engaging the rock surface and the techniques for installing the anchor system. Seafloor rock, category E of Table 1, will be filled in as the data need develops.

For each of the site categories, the performance of the various anchor types was determined. Thereby, those anchor types best suited for a given environment (and loading situation) were identified.

Loading

Horizontal. Given input information from some of the OTEC power plant proponents/concepts, CEL established upper limits to the range of horizontal load magnitudes to be considered in its anchor study. Because the present designs are all preliminary and because this study is to encompass all possible candidate power plant systems, the upper bound to required horizontal holding capacity was set at about twice the capacities required for present plant concepts. For the deep ocean site, static drag forces of 6 MN (1.27 x 10^6 lbs) and 9 MN (2 x 10^6 lbs) were estimated for two concepts: for such a site, the upper bound of the horizontal load capacity range for this study was set at 18 MN (4x10^6 lbs). For the Gulf Stream environment, the horizontal loading at the anchor was estimated at 67 MN to 71 MN (15x10^6) and (15.9x10^6 lbs) for two concepts: for the Gulf Stream site, the upper bound of the horizontal load capacity range has been set at 180 MN (40x10^6 lbs). Note that these load magnitudes are steady state due to wind and current drag forces. The load component at the anchor due to wave action on the floating structure is assumed to be completely damped out by the mooring line system.

Vertical. At the beginning of the CEL OTEC anchor effort a wide spectrum of mooring line angles and vertical mooring line force components was considered (Figure 2). Two loading envelopes were assumed. The first, representing the deep ocean, "benign" environment, assumed a maximum mooring line angle with the horizontal, β, of 1.40 rad (80 deg), with a corresponding maximum vertical load component of 100 MN (22.7x10^6 lbs). The high line angle of 1.40 rad was selected to include a proposed buoyant line design in which the deep section of the mooring line was to be highly positively buoyant rising at a steep angle from the anchor. The second loading envelope, representing the Gulf Stream environment assumed a maximum mooring line angle of 0.79 rad (45 deg) with a corresponding maximum vertical load component of 180 MN (40x10^6 lbs). Reference to these loading envelopes will be made throughout this report.

The authors do make note here that, at the time of writing, the high mooring line angles and the corresponding high vertical mooring line force components at the anchor do not appear practical. Perhaps the most significant drawback to the high line angles in a conventional mooring would be the increased stiffness of the moor and the resulting high dynamic load carried in the mooring line and transmitted to the anchor and supporting soil. Such high dynamic load acting on the predominantly calcareous oozes

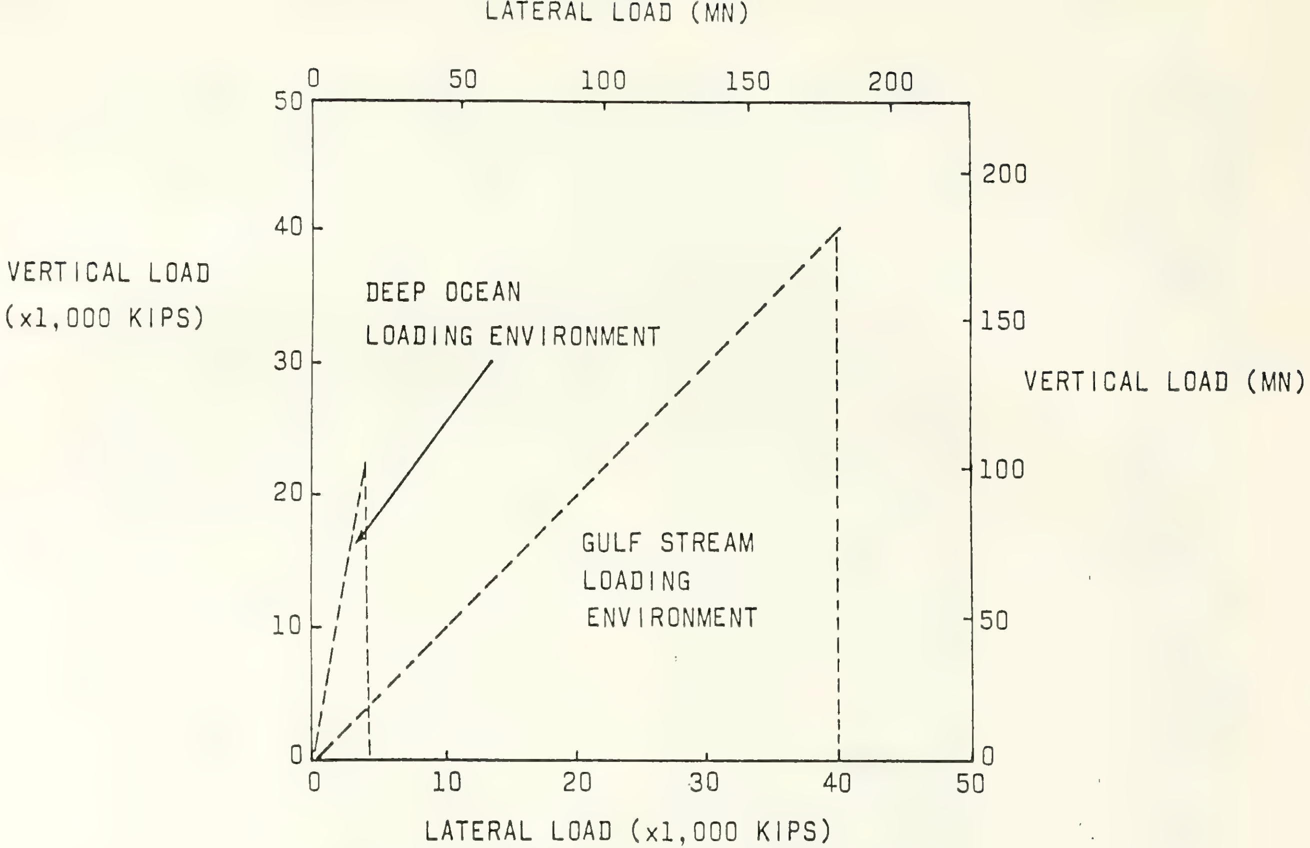


Figure 2. Assumed envelope of mooring line forces at the anchor (Jan 1976).

of the potential OTEC siting zones will result in significant soil holding capacity reductions. This necessitates increased anchor dimensions to reduce the critical stress levels. A better solution to the dynamic loading problem would seem to be to reduce the dynamic stress level by making the mooring more compliant by lowering the line angle.

STANDARD BURIAL ANCHORS

Introduction

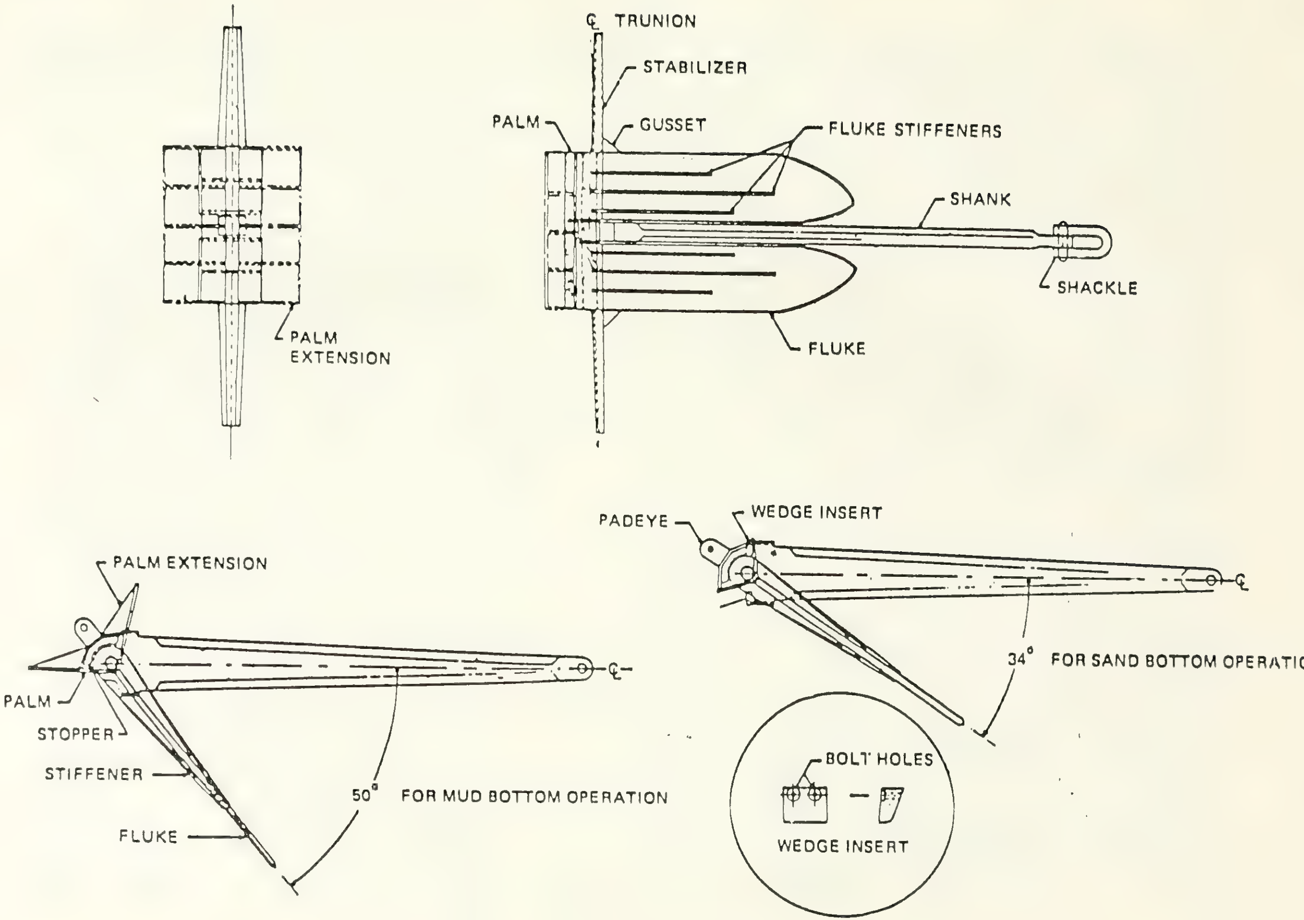
Standard burial anchors are a potential choice for anchoring structures of the sizes projected for OTEC. These burial anchors do not exist, however, in sizes large enough to yield holding capacities commensurate with simplified installation and efficient use of components. Use of the existing small anchors would result in very difficult load equalization problems. Overdesign would be required due to the high degree of uncertainty involved in achieving load equalization.

The largest anchor advertised available weighs 45 Mg (100,000 lbs); the largest anchor tested weighs 13.6 Mg (30,000 lbs); while the largest anchor tested sufficiently to yield data that could be used to extrapolate to sizes needed for OTEC was 6.8 Mg (15,000 lbs). This is a long way from the anchor weight required to yield holding capacities in the 13.3 MN to 44.5 MN (3×10^6 to 10×10^6 lb) range, the range which might be practicable.

Performance Prediction for Scaled-Up Anchors

Three basic types of burial anchors were chosen for extrapolation, the standard fluked type (e.g., STATO DANFORTH, BOSS), the pick type (e.g., BRUCE, HOOK, ADMIRALTY MOORING), and the mud type (e.g. DORIS, PARAVANE) anchors. Using relationships between anchor weight and anchor holding capacity (Valent et al., 1976) the required weights of these anchors could be determined for various soils and holding capacities. Extrapolation can be accomplished only for: (1) geometrically similar anchors, or (2) varying anchor geometries where the effect of variation on holding capacity is known. In order to provide some measure of confidence in the results of the extrapolation, geometric similarity will be maintained.

Fluked Type Anchor. The STATO configuration (similar to OFFDRILL, MOORFAST, STAYRITE) will be scaled-up because the majority of available data exists for this particular anchor. Figure 3 details the characteristics of the STATO anchor. If the existing STATO anchors were geometrically similar, the efficiency of the anchor type would be constant (Coombes, 1931). The data show, however, that efficiency decreases slightly with size. This occurs because the linear dimensions of each anchor do not vary exactly as weight to the one-third power, i.e. $W^{1/3}$. The


TRUNION
STABILIZER
PALM
GUSSET
FLUKE STIFFENERS
SHANK
SHACKLE
FLUKE
PALM
EXTENSION
PADEYE
WEDGE INSERT
34° FOR SAND BOTTOM OPERATIC
BOLT HOLES
WEDGE INSERT
PALM EXTENSION
PALM
STOPPER
STIFFENER
FLUKE
50° FOR MUD BOTTOM OPERATION

requirement that mild steel be used for fabrication caused plate thickness to increase as about $W^{1/2}$ for the larger sizes. For purposes of this evaluation, the 5.5 Mg (12,000 lb) STATO, which exhibits efficiencies at maximum drags of 25/1 in soft clay and 35/1 in sand, will be used as the basis for this extrapolation. The lattter value was controlled by steel stress rather than soil strength.

Geometric similarity is maintained by increasing steel strengths as well as the linear dimensions in proportion to $W^{1/3}$. Characteristics of the scaled-up anchors are given in Table 2. Anchor weights required to produce these capacities are shown in Figure 4. The required weight in the lower capacities is dependent upon soil strength; wehreas, anchor steel stress controls weight in the larger capacities. It is appparent from Figure 4 that once the criterion of limiting steel stress is achieved, anchoring efficiency no longer remains constant. Data from tests on existing anchors indicate that holding capacity increases in proportion to $W^{.69}$. This is extremely close to the theoretical limit of $W^{2/3}$. This slight difference is atrributed to errors in transferring data from log-log plots of actual steel stress versus anchor weight. Based upon this behavior, the most efficientanchor occurs at the intersection of the curves plotted for soil type and steel yield stress.

Another anchor, included in this category but somewhat different than the movable fluke type previously extrapolated, is the BOSS anchor, a fixed solid fluked anchor. Existing data show this to be a very efficient anchor in small sizes; however, there was insufficient data to confidently extrapolate performance. A conservative approximation of holding capacity was thus proposed.

$$R = 610\, W^{.75} \tag{1}$$

where R = holding capacity (N)
W = weight (N)

The exponent ".75" was recommended by Cole and Beck, 1969. The extremely high efficiencies in the small sizes is attributed to the thin, high strength sections used in the anchor. Without modifying the design by adding stiffeners on the flukes, the practical limit of this design, maintaining geometric similarity, is probably much less than 45 Mg (100,000 lbs). By adding stiffeners to this anchor, the performance should more closely resemble STATO type designs since fluke/shank angles and effective fluke areas are approximately similar. A primary advantage of the BOSS anchor is its ability to embed in competent seafloors, a problem often encountered with other fluked anchors. The technique of fixing the fluke on one side of the shank may be appropriate for the movable fluke anchors, if they are used in competent seafloors.

A pertinent characteristic of the enlarged fluke anchors is their expected penetration depth. This information is needed to enable design of the anchor pendant in contact with the seafloor and provide an idea of how much lateral movement is necessary to fully set the anchor. A reasonable approximation in clay is that the setting distance to achieve peak capacity is six times penetration depth. For example, penetration

Table 2. Characteristics of Existing and Extrapolated STATO Anchors (refer to Figure 3)

Nominal Weight	Anchor Dimensions in Meters							
	Fluke			Shank		Palm Extension		Stabilizer
Mg/Kips	length	width	thickness	length	width	length	width	length
5/12 1/	2.74	0.76	0.051	4.72	0.015	1.77	0.46	4.91
9/20	3.26	0.91	0.061	5.61	0.018	2.07	0.55	5.94
18/40	4.08	1.16	0.076	7.07	0.023	2.62	0.70	7.47
54/120	5.91	1.65	0.109	10.15	0.033	3.78	1.01	10.76
91/200	7.01	1.95	0.130	12.07	0.038	4.48	1.16	12.77
181/400	8.84	2.47	0.163	15.21	0.048	5.64	1.46	16.09
454/1000	11.98	3.32	0.221	20.63	0.066	7.65	1.98	21.85

1/ Existing anchor

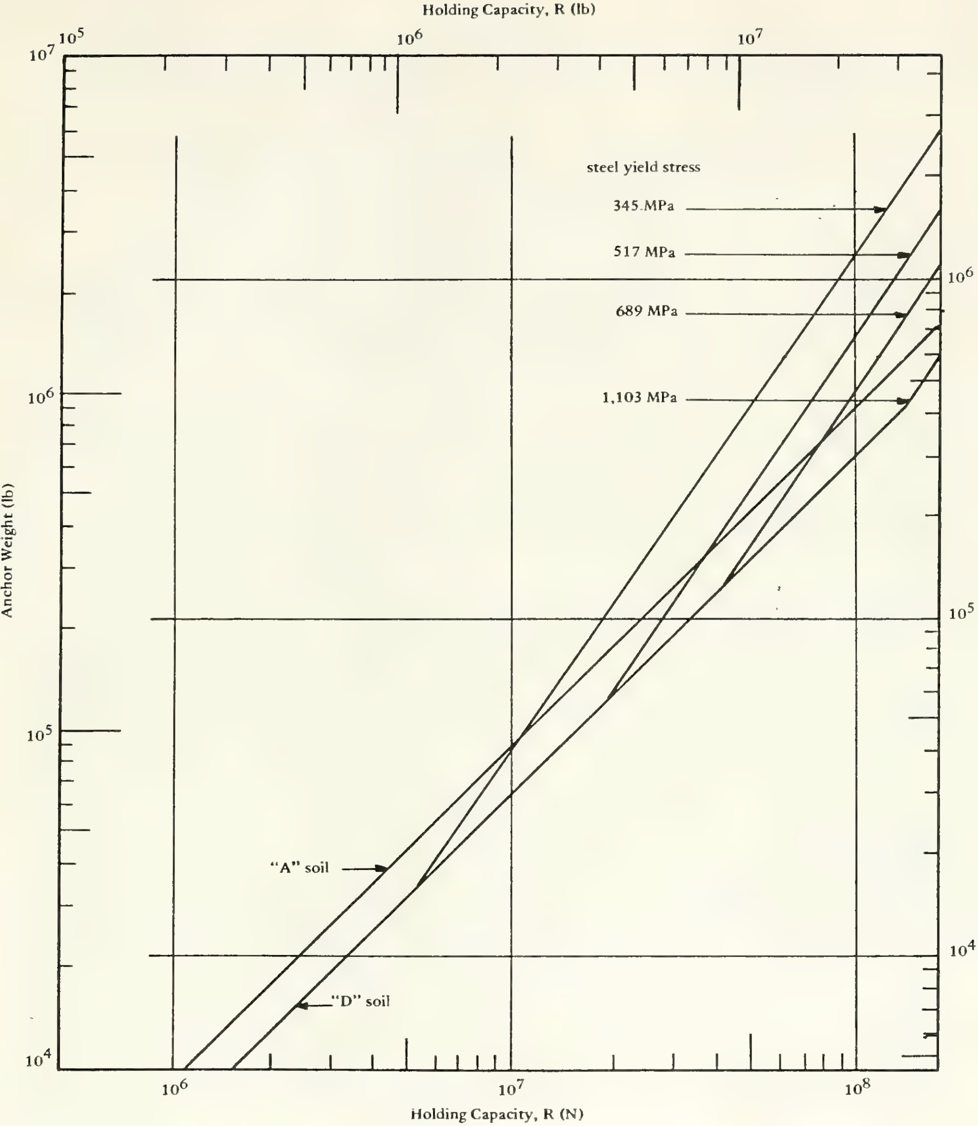


Figure 4. STATO anchor weights required to produce desired holding capacities.

depths in soil category A are found as follows:

(1) Computed required soil strength using anchor dimensions (Table 2) and peak design capacity.

(2) Obtain depth for required soil strength from shear strength versus depth curves (Valent et al., 1976).

(3) Plot depth versus anchor weight (see Figure 5).

The required penetration of a 45 Mg (100,000 lb) anchor would be 33.5 (110 ft) and total setting distance would be about 201 m (660 ft). Also plotted in Figure 5 is the actual penetration data for the STATO anchors in San Francisco Bay mud; the strength of this mud is very similar to category A soil. The remarkable agreement between actual and predicted penetration supports the extrapolated curve.

Pick Type Anchor. The BRUCE and HOOK anchors are pick type anchors. Relatively recent developments, they appear equally efficient (about 20/1 in category A soil and 28/1 in category D soil).

More data are available for the BRUCE anchor and it will, therefore, be used for extrapolation. The BRUCE anchor has one inherent advantage over the HOOK anchor in that it will always embed when pulled laterally, even if it lands on the shank side during deployment. This greatly simplifies deep water deployment.

Characteristics of the BRUCE anchor taken from company literature (Bruce, Ltd., 1974) are presented in Table 3 along with those of several extrapolated anchors to 455 Mg ($1x10^6$ lb) size. The 455 Mg anchor, if it could be fabricated, would yield 89 MN ($20x10^6$ lbs) holding capacity. The limiting characteristic, as with the STATO type, will be steel stress. Maximum holding capacity is plotted versus anchor weight for category A and D soils and for various limiting steel stresses in Figure 6. The steel currently used for the BRUCE anchor has a yield stress of 500 MPa (72.5 ksi which is greater than that required for the existing 6.4 Mg (14,000 lb) anchor. Maximum stress at maximum holding capacity for the 6.4 Mg anchor in the shank is 21 MPa (30 ksi) in soft clay (category A) and 29 MPa (42 ksi in sand (category D). From Figure 6, assuming a construction capability exists, it is apparent that approximately a 17.8 MN ($4x10^6$ lb) capacity 91 Mg (200,000 lb) anchor could be fabricated using exactly the same steel that is used in the 6.4 Mg (14,000 lb) anchor by simply scaling-up the dimensions in proportion to weight to the one-third power, i.e., $W^{1/3}$.

Also included in Figure 6 is a curve representing the theoretical relationship between holding capacity and anchor weight in that regime where steel stress control efficiency, i.e., holding capacity ≃ $(\text{weight})^{2/3}$ The initial point of the curve came from the company advertised limiting load of 2.98 MN (670,000 lbs) for the 6.4 Mg anchor fabricated from 500 MPa steel.

Mud Type Anchor. Only two of this type anchor are known to exist and are called the PARAVANE anchor and the DORIS Mud anchor. The existing sizes and company advertised capacities of the PARAVANE or "Kite" anchor are provided in Table 4. These capacities appear to be based upon a soil

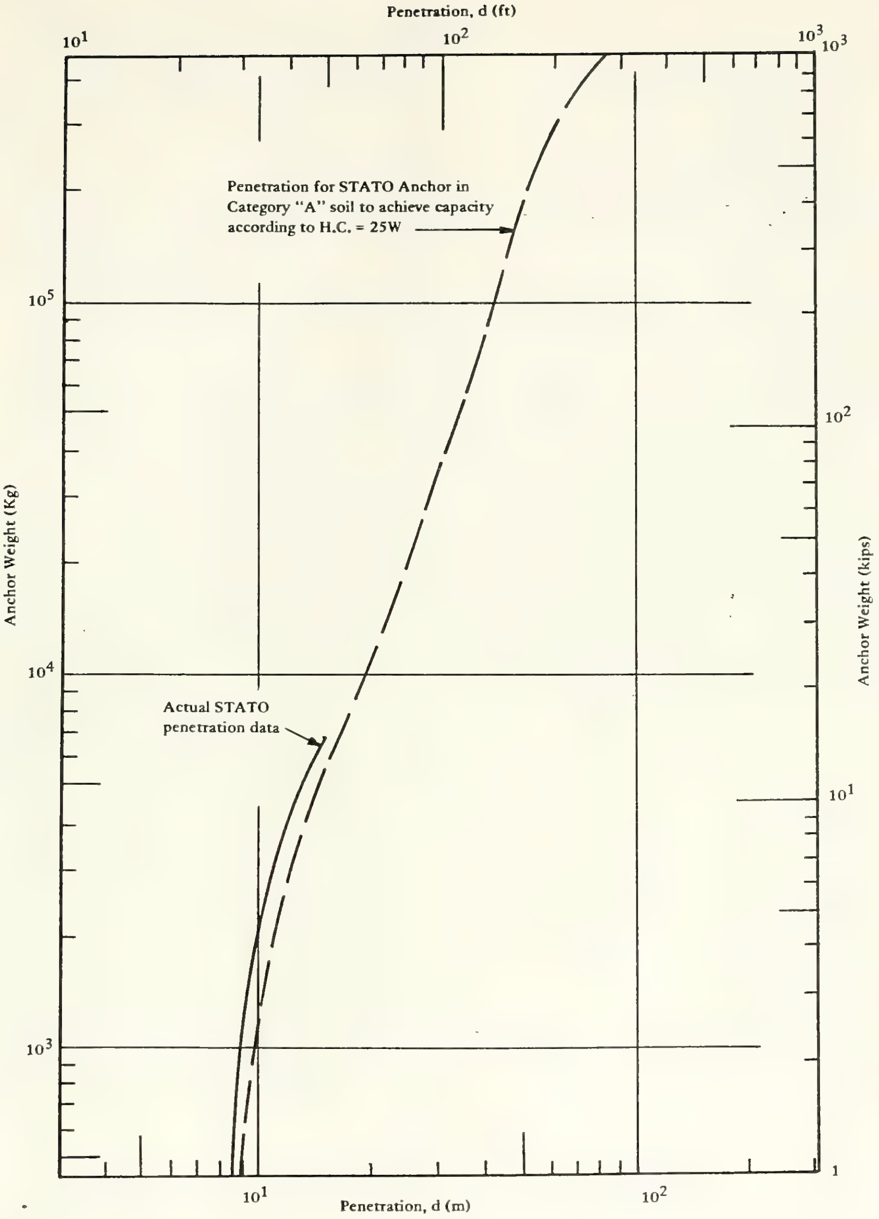
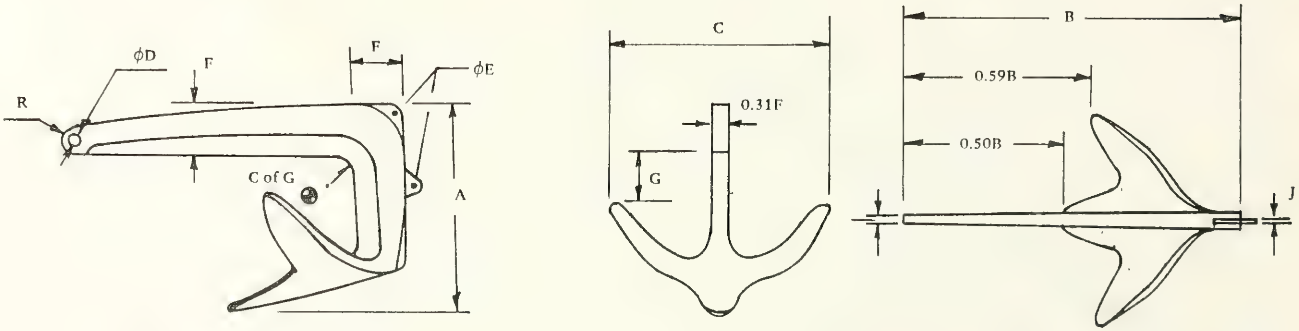


Figure 5. Predicted penetration of the STATO type anchor versus anchor weight.

Table 3. Characteristics of Extrapolated BRUCE Anchors

Nominal Wt. Mg/kips	A(m)	B(m)	C(m)	D(m)	E(m)	F(m)	G(m)	H(m)	J(m)	R(m)	RR(m)
6.2/14	2.56	4.54	2.90	0.10	0.20	0.67	0.53	0.12	0.09	0.33	0.14
9.1/20	2.37	5.12	3.26	0.11	0.23	0.75	0.60	0.13	0.10	0.43	0.15
18.1/40	3.63	6.43	4.11	0.14	0.29	0.94	0.75	0.17	0.13	0.54	0.19
54.4/120	5.21	9.30	5.91	0.20	0.41	1.36	1.09	0.24	0.18	0.73	0.28
90.7/200	6.19	11.03	7.01	0.24	0.49	1.62	1.29	0.29	0.22	0.92	0.33
181.4/400	7.80	13.90	8.34	0.30	0.62	2.03	1.62	0.37	0.27	1.16	0.42
453.6/1000	10.58	18.84	12.01	0.41	0.84	2.76	2.20	0.50	0.37	1.58	0.57



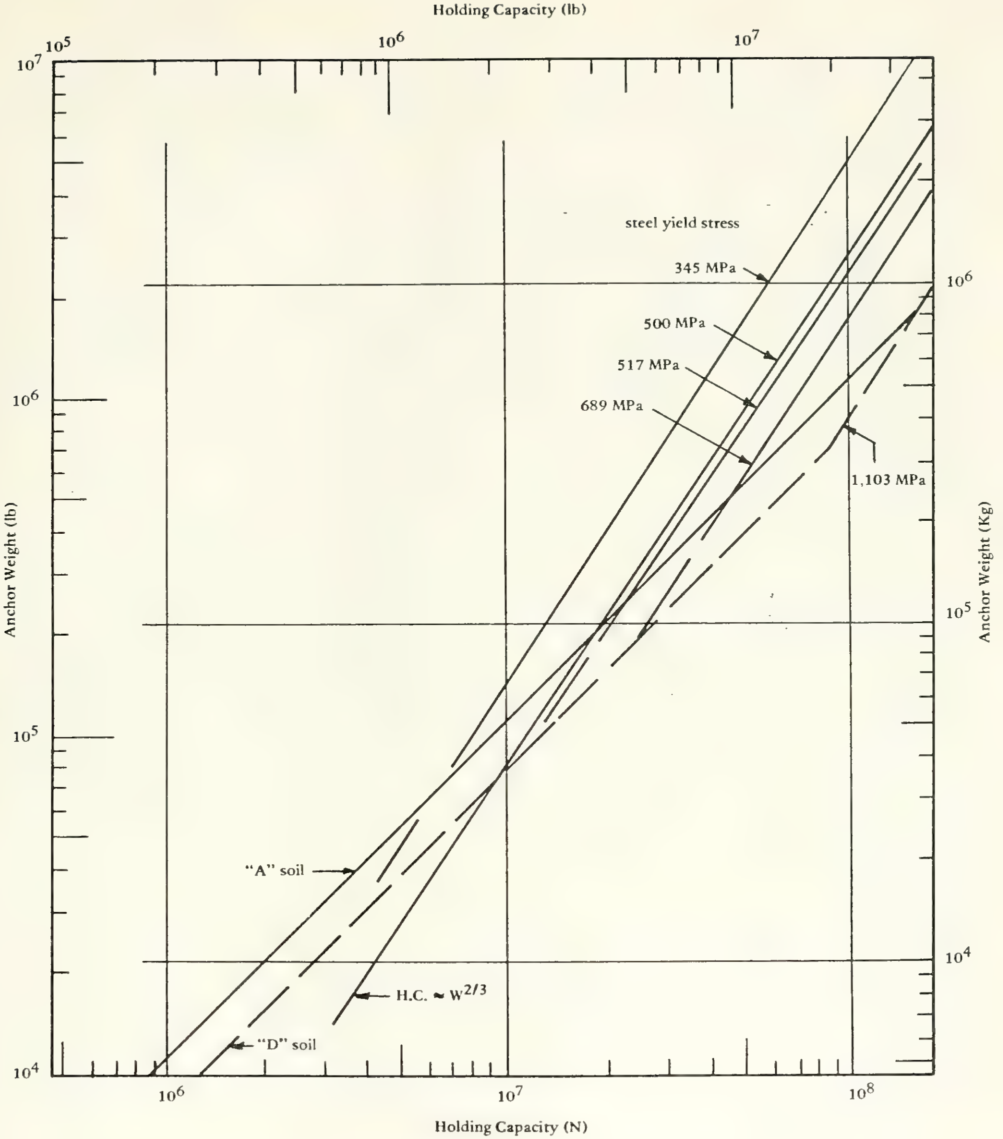


Figure 6. Bruce anchor weights required to produce desired holding capacities.

Table 4. Characteristics and Company Advertised Holding Capacities of the Paravane Anchor

		Model No.				
		200	500	1000	3000	6000
Weight kg (lb)		94.8 (209)	225.4 (497)	449.1 (990)	1420.7 (3132)	2717.0 (5990)
size, m	h	0.73	1.00	1.26	1.81	2.29
	w	0.98	1.34	1.69	2.44	3.09
	l	0.91	1.23	1.56	2.34	2.83
		Holding Capacity, kN/kips				
Embedment Depth, m	1.5	36/8	44/10	80/18	151/34	222/50
	3.1	53/12	93/21	138/31	302/68	445/100
	4.6	80/18	129/29	200/45	423/95	667/150
	6.1	98/22	178/40	285/64	583/131	898/202
	7.6	--	222/50	351/79	725/163	1125/253
	9.1	--	--	409/92	872/196	1352/304
	10.7	--	--	--	1023/230	1579/355
	12.2	--	--	--	--	1801/405
	13.7	--	--	--	--	2024/455
	15.2	--	--	--	--	2246/505

with a shear strength approximately three times as strong as category A soil. This design does not lend itself to significiant extrapolation due to the high bending stress created at the juncture of the plates. Rough estimates indicate that the maximum practical size would be in the 13.6 Mg (30,000 lbs) to 18 Mg (40,000 lbs) range with a capacity in category A soil of about 1.3kN to 1.8 kN (0.3 to 0.4×10^6 lbs).

Data on the DORIS Mud Anchor (C.G. Doris, 1973) are provided in Table 5. Advertised holding capacities are theoretically predicted; they are not based upon test data. In order to achieve these holding capacities with the shear strengths suggested by C.G. Doris (3.4 kPa to 9.6 kPa) the anchors must embed to depths approximately 2 1/2 times their height. Data would be needed to verify this penetration before confidence could be placed in the performance of the extrapolated anchors.

The DORIS anchor appears to be suitable for very soft sediments where soil shear strength is relatively uniform or increases very slowly with depth. In normally consolidated soils like category A soil, there is a definite advantage to deep penetration. Bearing area for the DORIS anchor is roughly 1 1/2 times that for comparably weighted fluked and pick anchors, yet penetration of the latter would seem to be much greater. This would more than offset the area advantage of the DORIS. This is substantiated by the difference of more than a factor of 2 in efficiencies between the fluked or pick types and the DORIS anchor.

Large anchor-bearing areas are employed for the DORIS anchors to achieve high capacities with shallow burial. This results in low steel stress. The advantages of the DORIS anchor should be more fully realized in the very high load ranges where the fluked and pick type anchors become steel stress limited. The point at which this occurs in the DORIS anchor cannot be determined because plate thicknesses and steel types are unknown.

Extrapolation of the DORIS anchor to the ultra large sizes required by OTEC cannot be made with a great deal of confidence by maintaining geometric similarity. Extrapolation on this basis assumes that anchoring efficiency remains constant, until limited by steel stress. As dimensions increase by $W^{1/3}$, bearing area increases as $W^{2/3}$. Thus, for constant efficiency, penetration must increase. According to company data (Table 5), bearing area increases as $W^{6/7}$ to maintain constant efficiency (i.e., not geometrically similar). This means that fluke thickness is increasing at a rate less than $W^{1/3}$ which results in a more rapid increase in steel stress than would occur if similarity were maintained. In other words, the company probably assumed that penetraton would not increase enough to maintain constant efficienty. Therefore, the increase in bearing area had to be disproportionately large (larger than $W^{2/3}$); and thickness relatively small. It is apparent that this offsets the advantage of maintaining low bearing pressure to produce low steel stress. Even though extrapolation is uncertain, the general size of some large anchors have been calculated and are presented in Table 6 solely to enable estimates of fabrication and logistics difficulties. Assuming geometric similarity is not maintained, bearing areas and holding capacities were scaled according to $W^{6/7}$ and $23W^{.9}$, respectively (Valent et al., 1976). The ratio of fluke height to

Table 5. Characteristics of Standard DORIS Mud Anchors(DORIS, 1973)

Type	Weight Mg/kips	Dimensions Meters			Advertised Holding Capacity kN/kips
		Length	Width	Height	
A20	1.5/3.3	3.00	2.80	1.00	196/44
A28	2.2/4.8	3.80	3.20	1.30	276/62
A40	3.5/7.7	4.40	4.00	1.50	391/88
A70	6.0/13.2	5.50	5.00	1.90	685/154
A110	9.0/19.8	6.60	6.00	2.20	1076/242

Table 6. Characteristics of Extrapolated DORIS Mud Anchors

Anchor Weight Mg/kips	Length (m)	Width (m)	Height (m)	Holding Capacity kN/kips	Efficiency
9.1/20	6.52	5.94	2.22	1080/240	12.1
18.1/40	8.81	7.99	2.99	1960/440	11.0
54.4/120	14.26	12.95	4.85	5250/1180	9.8
90.7/200	17.59	16.00	6.00	8450/1900	9.5
181.4/400	23.77	21.61	8.11	16,000/3590	9.0
453.6/1000	35.02	31.82	11.95	36,000/8100	8.1

width was taken as 3/8.

The low efficiencies and large sizes of the DORIS compared to the fluke and pick type anchors previously discussed, seem to rule out this choice for OTEC applications. For example, assuming it could be fabricated, a 455 Mg (1,000,000 lb) mud anchor would be required to equal the capacity of a 136 Mg to 182 Mg (300 to 400,000 lbs) fluke type anchor, and the large size, at least 36 m (118 ft) long, 33 m (107 ft) wide and 12 m (40 ft) high, would make the mud anchor extremely difficult to handle and deploy.

Anchor Fabrication

The limiting criterion regarding conventional anchor use in OTEC is fabrication capability. Two methods are available with which to fabricate these anchors, welding and casting.

The feasible construction limit for a welded anchor, without resorting to costly built up section, depends upon available steel. Heat treated steels of 760 MPa to 1100 MPa (110 to 160 ksi) strengths in thicknesses of .051 to 0.10 m (2 to 4 in.) can be obtained; however, the complexities involved with welding these materials make them undesirable choices. The most desirable steels for this application are HY-80 and A514. Of these, HY-80 is superior because of its superior toughness. Thicknesses of up to 0.11 m (4.5 in.) are available. This would allow a 68 Mg (150,000 lbs) fluke type anchor to be fabricated.

The feasible limit for a casting depends primarily upon the volume of steel required. Available information suggests that this would translate into approximately a 45 Mg to 64 Mg (100,000 to 140,000 lbs) maximum size for a pick or fluke type anchor. The practical limit for cast steel strength is 517 kPa (75 ksi). This value is almost coincident with the steel used to cast the Bruce anchor. One alternative is to fabricate an anchor by partial casting and welding. This would increase cost over a normal casting but could also potentiallly increase practicable size.

A significant factor in fabrication procedure is cost. For example, casting large anchors (45 Mg range) requires large flask capacity and would cost roughly $3.30 to $4.40 per kilogram including pattern fabrication. The cost of a welded construction of this size would be $5.50 to $6.60 per kilogram. The actual cost may be higher than the quoted figures because there would undoubtedly be some engineering involved in fabricating these large anchors. The largest anchor that has been fabricated in this country is 27 Mg (60,000 lbs) (April, 1976) even though the largest advertised is 45 Mg (100,000 lbs).

Cost is not the only criterion that should be used in selecting fabrication method. Fabrication and use experience are also significant. There is limited experience in casting large anchors; thus, it appears that there would be more uncertainty both with fabrication and in-service use. Quality control of a very large casting may be difficult. Thus, the lower limit of the potential range (45 Mg) is recommended for casting.

Summary

Several standard drag-burial anchors including fluke, pick, and mud types were evaluated to determine the maximum practicable sizes that could be fabricated, handled, and deployed. The maximum size, 45 Mg to 68 Mg (100,000 to 150,000 lbs), will be controlled strictly by fabrication capability. The capability to handle and deploy components of this magnitude is well established.

Table 7 summarizes the characteristics of the possible anchors for OTEC. Maximum recommended sizes are 68 Mg (150,000 lbs) for a welded fluke type anchor and 45 Mg (100,000 lbs) for a cast anchor. Casting is the only reasonable method for fabricating the Bruce (pick type) anchor since its shape is not amendable to welded construction. A 64 Mg (140,000 lbs) cast anchor is conceded as a possiblity within the present state-of-the-art with the realization that limited data exists in castings of this size. Anchor cost varies from 1 to 3 cents per newton (6 to 12¢ per pound) of holding capacity. In practice these differences would not be as significant because the higher cost is associated with a higher capacity anchor; thus, the number of mooring legs would be reduced with commensurate savings in materials and installation costs. Anchor size differences are not significant provided the fluke type anchor employs hinges in the stabilizer bars; otherwise, the approximately 12.2 m (40 ft) width would complicate deployment.

The drag burial anchors discussed are feasible to lateral capacities to 22 MN ($5x10^6$ lbs). The difficulties inherent in using standard drag anchors in combination makes this type unsuitable for the Gulf Stream sites where loads to 178 MN ($40x10^6$ lbs) are possible. It is very doubtful that eight of these large anchors could be placed in parallel such that each would be equally loaded.

Even in the deep water, benign environments where the lateral loads range only up to 18 MN ($4x10^6$ lbs), drag anchors are not desirable. This is true because the drag embedment anchor requires a zero mooring line angle with the seafloor in order to embed, and requires that a near-zero angle be maintained in order to realize best holding capacity. Also, the drag embedment anchor is able to resist load from only one direction. These requirements of near-zero mooring line angles and uni-directional loading are too restrictive for general OTEC application.

The above requirements for drag embedment anchors can be satisfied. For instance, the uni-directional loading requirement can be met by using a multi-point moor in which each individual leg and its anchor experience service load is essentially from one direction. The requirement of zero mooring line angle during anchor embedment can be achieved in at least three ways:

1. A sufficiently long scope of line could be used such that the mooring line angle at the seafloor was zero. However, the cost of the long line will be quite high, and the following techniques for shortening the scope will likely prove more cost effective.

Table 7. Summary of Characteristics of Possible Drag Embedment Anchors for OTEC

Anchor Type	Fabrication Method	Maximum Weight Suggested Possible	Holding Capacity MN/10^6 lbs		Holding Capacity Cost cents per N/cents per lb		Size (m)	
		Mg/kips	Cat A	Cat D	Cat A	Cat D	Length	Width
Fluke	Welding	68/150	16.9/3.8	22.2/5.0	2.7/12	2.0/9.0	10.67	3.96** 11.58
		45.4/100	11.1/2.5	15.6/3.5	1.8/8	1.4/6.0	9.14	3.35** 9.75
Fluke	Casting	63.5*/140	15.1*/3.4	21.4*/4.8	1.8/8	1.4/6.0	10.36	3.96** 11.28
		45.4*/100	8.9*/2.0	12.0*/2.7	2.3/10	1.7/7.5	8.53	5.49
Pick	Casting	63.5*/140	13.3*/3.0	15.6*/3.5	2.3/10	1.8/8.0	9.75	6.40

*Maximum Casting Size, potentially involves quality control problems.
**Width with stabilizer hinged and folded.

2. Deadweight may be added to the mooring line near the anchor to "absorb" the vertical component of mooring line load. The deadweight may be mass blocks attached to the mooring line or may be a length of heavy chain incorporated into the lower end of the mooring line.

3. The drag embedment anchor could be fully embedded during installation using a long scope of line and zero mooring line angle, and then the scope shortened considerably raising the line angle to 0.1 rad (6 deg). Once embedded, the drag anchor will hold near capacity at this 0.1 rad line angle. This approach presumes that it is possible to preset an anchor to a few million pounds. Developing the necessary lateral force may be possible by pulling one anchor against another, but it will be difficult and time consuming. Further, if the preset load was exceeded, the anchor could pull out.

Given that one of the above options for achieving a near zero line angle during embedment is invoked, an appropriate drag embedment anchor could be built. An anchor such as the Bruce would be desirable because it would reorient itself and embed regardless of initial attitude. Maximum lateral capacity would then be limited to 9-16 MN (2-3.5x10^6 lbs) without resorting to multiple anchor hookups.

Conclusions

Of the two anchor types considered potentially applicable to OTEC, the fluke and pick types, the least complicated installation would be achieved with the pick type because of its ability to embed irrespective of landing attitude. The maximum capacity of this anchor would be 9 to 16 MN (2 to 3.5 million pounds). If the more complicated installation procedures associated with the fluke type are acceptable, then 13 to 22 MN (3 to 5 million pounds) capacity is possible. For these anchors to be useful, they would have to be used in multi-point moorings possibly with several anchors in tandem or parallel in each leg.

Until fabrication technology improves to a point where 114 Mg to 182 Mg (250,000 to 400,000 lbs) anchors are feasible, the use of conventional anchors of the pick and fluke types would seem to be less attractive than other types of anchors for the OTEC installation.

PLATE ANCHORS

Introduction

Several plate anchors were designed and analyzed for holding capacity. Calculations were made for seafloor soil categories A, C, and D. Possible OTEC plate anchor designs were also compared to existing plate anchor technology. Rough cost estimates were made on the basis of present material and fabrication costs.

Holding Capacity

Soil categories A and C (cohesive soils) were analyzed for both long term and short term behavior under static loading. Soil category D (cohesionless) was analyzed for long term static loadings. Anchors considered were flat plates with areas from 9.3 m^2 to 149 m^2 (100 ft^2 to 1600 ft^2). Calculations were made for both square and rectangular (length = 2 x width) plates. Holding capacities at anchor embedment depths of 6.1 m to 30.5 m (20 ft to 100 ft) were determined. The procedure proposed by Taylor and Lee (1972) was used.

Short Term Holding Capacity. Short term static loading describes the situation in which the anchor is loaded rapidly until breakout occurs. For short term loadings the holding capacity to be achieved is calculated differently for cohesive soils (clays) than for cohesionless soils (sands).

In cohesive soils the excess soil pore pressure generated by rapid loading acquires a lengthy time period for near-complete dissipation. Because the pore pressure beneath an embedment anchor plate does not have sufficient time to dissipate during rapid loading, suction develops under the plate causing an apparent increase in holding capacity. The magnitude of this suction force cannot be predicted confidently for the highly disturbed soil expected about the large OTEC plate anchors. Because of this lack of confidence, the contribution of the suction force to short-term holding capacity was herein conservatively assumed to be zero; i.e. no suction.

In cohesionless soils the excess soil pore pressure due to rapid loading is dissipated almost as fast as it is generated. Thus, a suction force is not generated.

The basic equation used for holding capacity is a typical bearing capacity equation (Taylor et al., 1975):

$$R = A(s\bar{N}_c + \gamma d\bar{N}_q)(0.84 + 0.16\ B/Z) \qquad (2)$$

where R = holding capacity (N)

A = plate area (m^2)

s = soil cohesion (Pa)

γ = buoyant unit weight of soil (N/m^3)

d = plate embedment (m)

$\bar{N}_c$, $\bar{N}_q$ = holding capacity factors

B = fluke width (m)

Z = fluke length (m)

For cohesionless soils, s = 0 and $\overline{N}_c = 0$. For cohesive soils under short term leading, $\overline{N}_q = 1$ and $\overline{N}_c$ ranges from a value of zero to nine.

Long Term Holding Capacity. Long term static holding capacity refers to the situation in which an anchor pulls out after a constant upward force has been applied over a long period of time. Such a situation might occur for a submerged buoy moored to the seafloor (Taylor et al., 1975). Equation (2) with s and $\overline{N}_c$ set to zero was used to find long term capacity. $\overline{N}_q$ ranged from two to twelve. Under long term loading, the excess pore pressure in cohesive material escapes (drained behavior). As a result, clays under long term loading act as frictional materials. Cohesionless materials were assumed to exhibit drained behavior only.

Design Holding Capacity. Short and long term holding capacities were plotted against depth and plate size in Figures 7, 8, and 9. For cohesive soils the short term capacity was much less than predicted long term capacities. Therefore, short term capacity was taken as the critical or design holding capacity. The curves show the large plate sizes and deep embedment depths required to achieve OTEC holding capacities. The single plate size required in the deep ocean environment, soft clay (categeory A), is 12.2 m wide. Compare this with the largest existing embedment anchor plate which measures 1.5 m wide to 2.5 m long.

Effect of Inclined Load. Another consideration that enters into a single point mooring (single or multiple flukes) is the effect of inclined or non-vertical loads. Several studies (Meyerhof, 1973; and Colp and Herbich, 1972) on the effects of inclined loads have been conducted. For almost all soil types and relative depths of embedment Meyerhof found that holding capacity under an inclined load equalled or exceeded that under a vertical load. Tests by Colp and Herbich in a saturated sand showed that capacity increased up to an angle of .44 rad (25 deg) and then decreased to near the vertical capacity at an inclination of 0.79 rad (45 deg) vertical. For this report the effect of load inclination changes was assumed to be negligible.

Operational Factors

Mooring Line Angle. For a given horizontal load at the surface, the load felt at the anchor is a function of the mooring line angle (angle between seafloor and mooring line). For a taut line the load at the anchor increases approximately as the secant of the line angle at the seafloor. Thus, for high line angles mooring loads are increased significantly. For example, a horizontal force of 18 MN (4×10^6 lbs) would produce a force in the mooring line about 25 MN (5.7×10^6 lbs) at a line angle of $\pi/4$ rad (45 deg). At an angle of 1.4 rad (80 deg) the force in the mooring line would be roughly 102 MN (23×10^6 lbs). The advantage of maintaining a low mooring line angle is apparent.

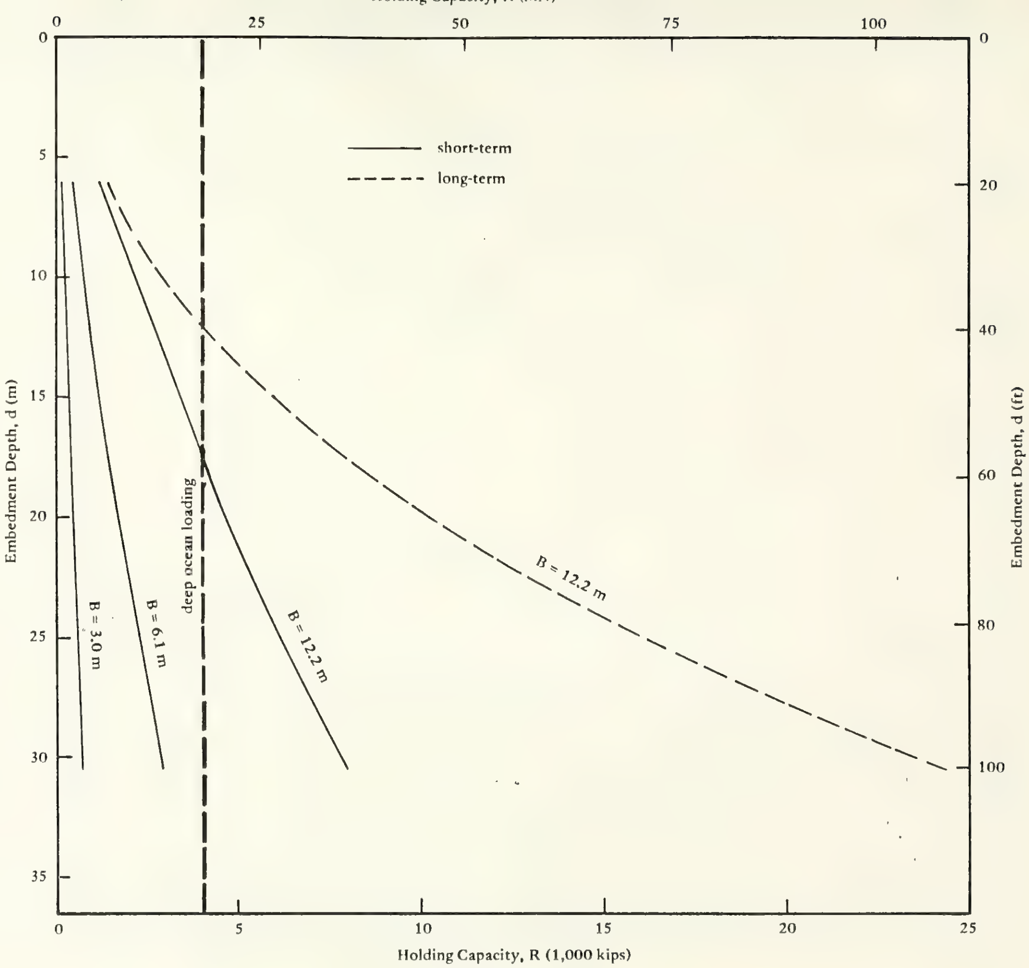


Figure 7. Holding capacity of square plate anchors in soil category A.

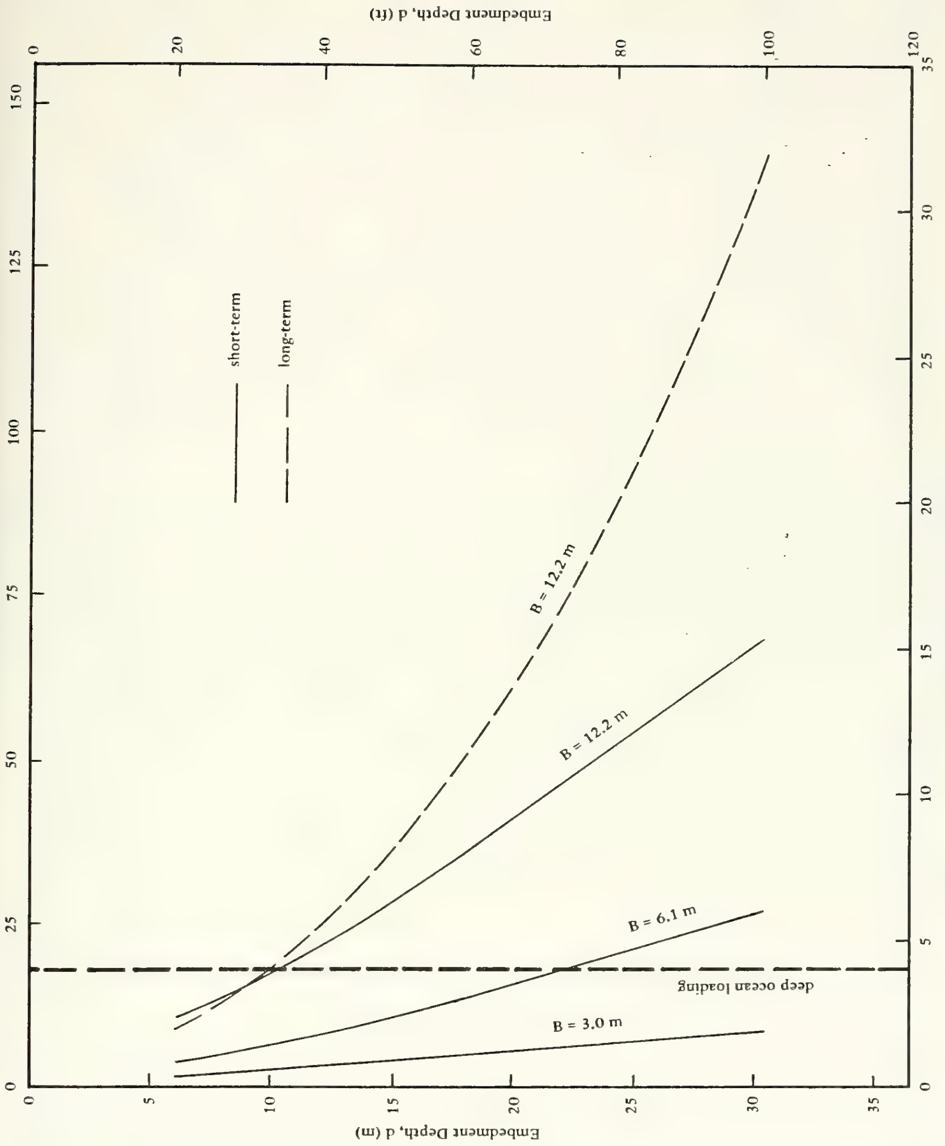


Figure 8. Holding capacity of square plate anchors in soil category C.

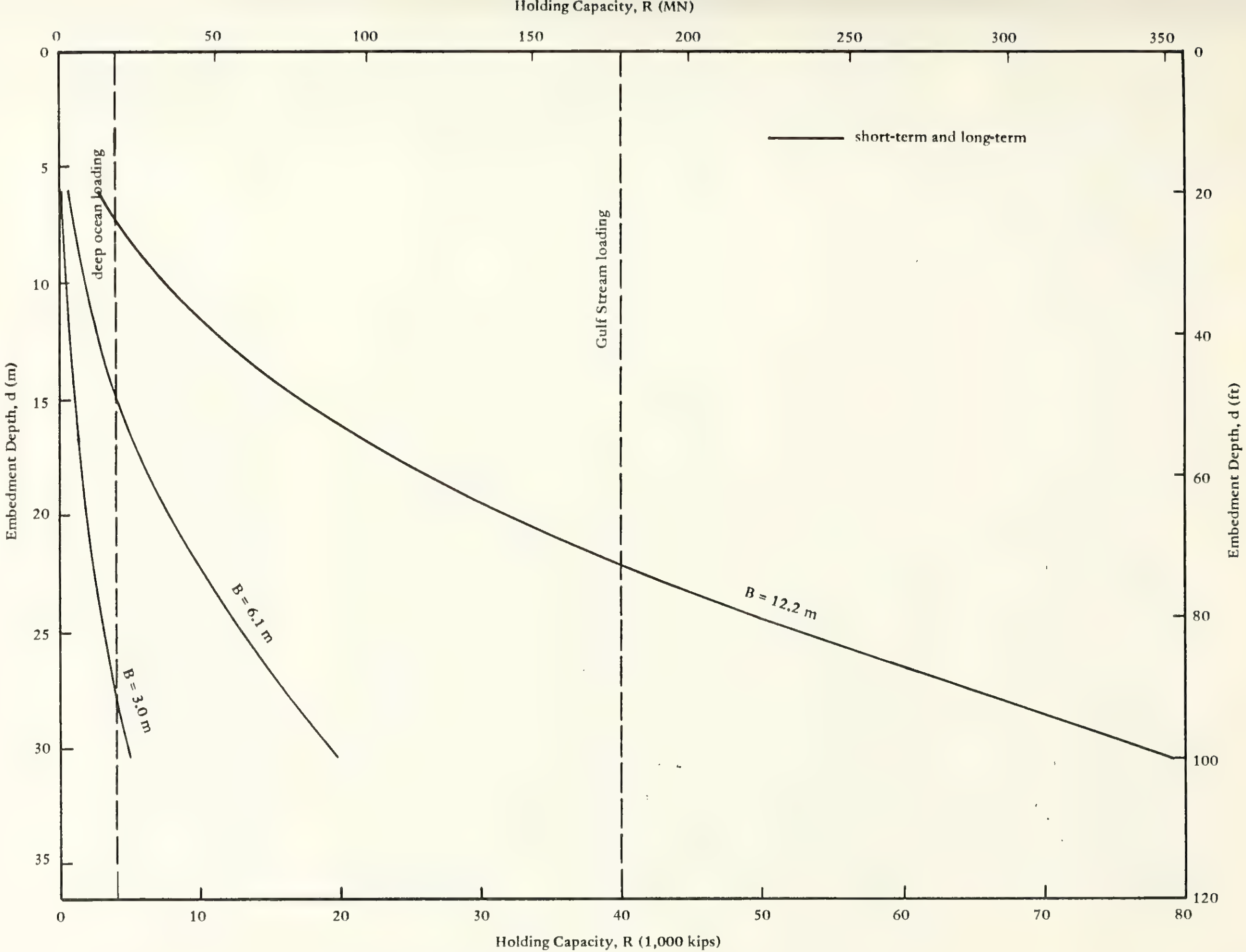


Figure 9. Holding capacity of square plate anchors in soil category D.

Keying Distance. Keying distance is the vertical distance (upward) required to rotate the anchor to an approximately horizontal orientation (Figure 10). A rule of thumb is that the keying distance equals twice the fluke length. For example, suppose that a 6 m wide fluke must be embedded to 20 m to mobilize soil resistance. To account for keying distance, the fluke length, i.e., required penetration = 20 m + 2 x 6 m = 32 m. For typical OTEC relative embedments (d/Z = 5) this distance becomes excessive. The large plates must be driven very deep to account for keying distance.

Single Plate. Design anchor loadings (Table 8) were used with Figures 7, 8, and 9 to specify required OTEC plate anchor sizes. The results are listed in Table 9. Table 9 suggests that a single plate anchor could be used to moor OTEC. The basic advantage of these single fluke designs is their simplicity. Note, however, the size and embedment depth required. These sizes represent an enormous extrapolation of current plate anchor installation technology. The means to embed and key such anchors neither exists today nor appears possible in the foreseeable future.

Multiple Plate. As shown in Table 9, required holding capacities for OTEC could be obtained by bridling several flukes. This approach introduces the problem of load equalization among anchors. A simple sheave and cable system (Figure 11) has been used to solve this problem for two anchors. But designing, installing, and maintaining a bridling system for up to six anchors would be considerably more intricate and costly. Note also that the driven depth for 3 m flukes in category A soil is 37 m. The Civil Engineering Laboratory has explosively embedded its 100 kip anchor to depths of 15 m in soft seafloors using a 254 mm (10 in.) inside diameter gun. By increasing gun diameter to 406 mm (16 in.), the 3 m plate could probably be embedded to only about 15 m. Although other means of embedment are possible (vibrating, jetting, etc.). experience indicates that they would be even less desirable.

The load equalization and embedment problems associated with multiple plate anchor installations would require a concentrated and expensive design and development program with no guarantee of success. Such a program is not recommended in view of the alternative anchor types available.

Structural Design

Method. The simple model shown in Figure 12 was used to compute stresses in plate anchors. Mooring force was assumed to act vertically. It was balances by a uniform distribution of soil pressure on the horizontal plate. Maximum bending moments and resulting fiber stresses were calculated and used to compute required steel thickness. Under the assumptions, required plate thickness is independent of plate area. A plot of mooring force versus required plate thickness (HY-80) steel is shown in Figure 13. From the plot it is clear that steel of 76 mm to 101 mm (3 to 4 in.) thickness is required.

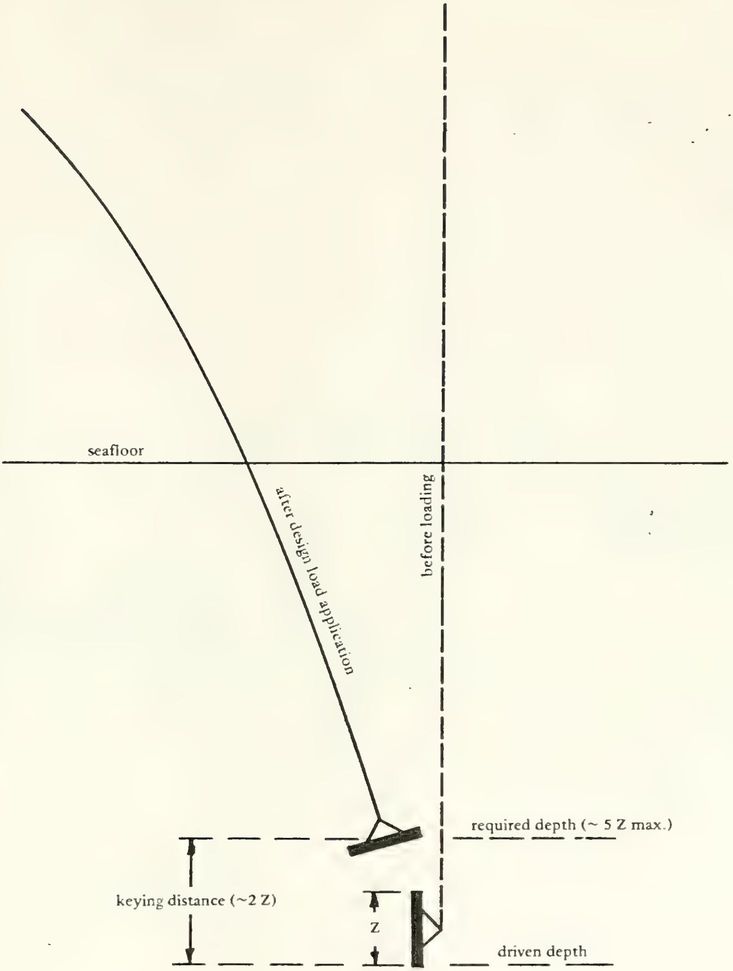


Figure 10. Illustration of keying distance.

Table 8. OTEC Anchor Loading

	Deep Ocean	Gulf Stream
Possible Soil Category	A, B, C	D, E, F
Assumed Maximum Mooring Line Angle with Seafloor	0.79 rad (45 deg)	
Maximum horizontal force	18 MN (4×10^6 lbs)	180 MN (40×10^6 lbs)

Table 9. Plate Anchor Design for OTEC Application

Load MN/10^6 lbs	Soil	Width [1] (m)	Thickness (mm)	Required Depth (m)	Driven Depth (m)	Number [2] Required
25.5/5.6	A	3.05	83	30	37	9
		6.10	142	30	43	2
		12.19	203	24	48	1
25.5/5.6	C	3.05	142	30	37	4
		6.10	203	30	43	1
		12.19	203	14	38	1
255/56	D	3.05	229	30	37	12
		6.10	457	30	43	3
		12.19	668	26	50	1

1/ Square in Plan

2/ Factors of Safety not included

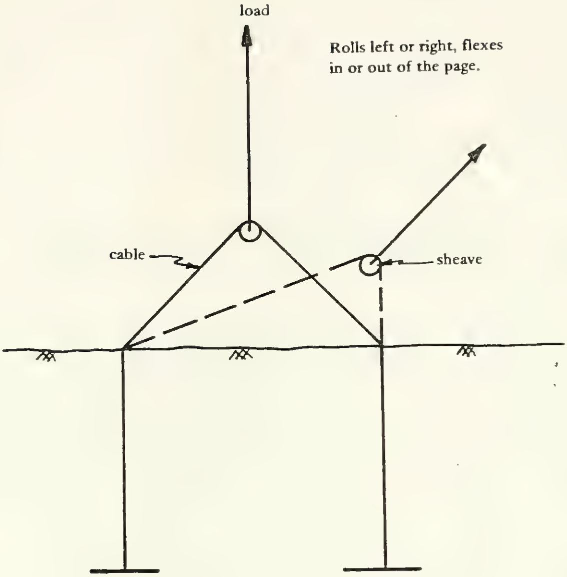


Figure 11. Load equalization for two anchor case

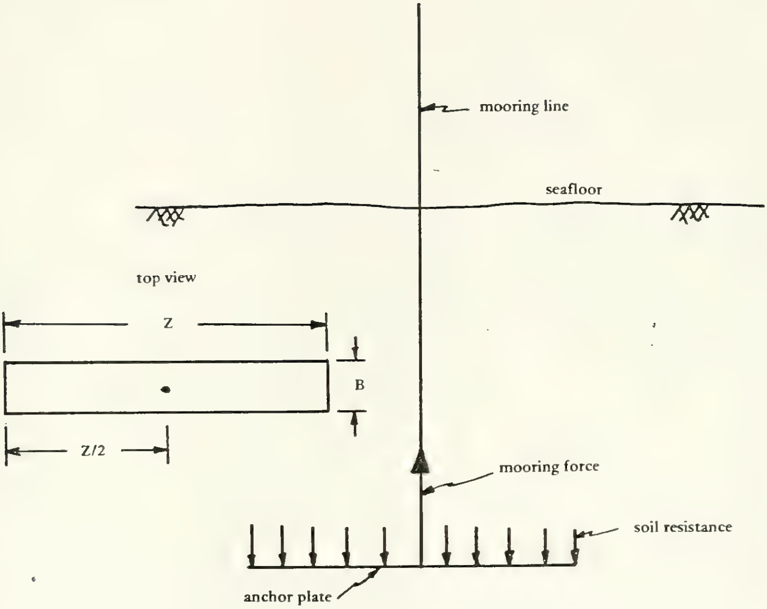


Figure 12. Model for plate anchor structural design.

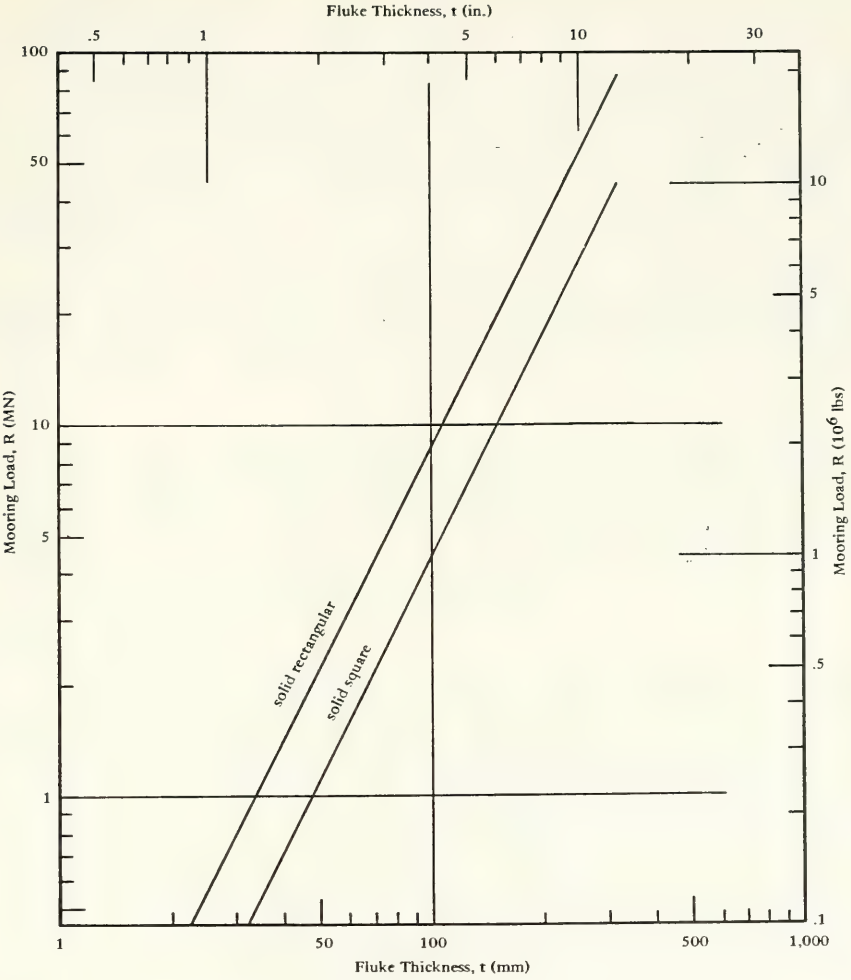


Figure 13. Required fluke thickness versus mooring load.

To reduce the steel weight (cost) imposed by large plate thicknesses, a hollow plate design was also considered. Typical solid and hollow designs are shown in Figure 14.

Anchor loading was plotted against anchor weight for both solid and hollow designs in Figure 15. The reader should note that this figure does not indicate anchor holding capacity. It merely gives the weight (thickness) required to prevent steel failure under an applied load. Anchor capacity will be a funciton of soil type and embedment depth as shown in Figures 7, 8, and 9. The nine square metre (100 ft^2) anchor has a maximum holding capacity of 3.3 MN (0.75×10^6 lbs). Now Figure 15 is entered to determine anchor weight.

Cost. Table 9 shows several possible anchor designs for OTEC applications. The 3 m square anchor was chosen for cost comparison to present anchors. The estimated cost of fabrication and materials only for a deep ocean plate anchor installation (nine 3-m anchors) is $234,000 (Table 10).

Summary

Plate anchors, either singly or in arrays are not reasonable choices for OTEC. Load equalization for multiple fluke designs and embedment limits for either multiple or single fluke design are critical. In view of the alternative anchors available, further investigation of plate anchors is not justified.

Existing or moderately scaled up plate anchors could be useful in conjunction with another primary anchoring system.

Screw-In Plate Anchors

Present types of screw anchors consist of small diameter steel shaft with a large diameter plate, or blade, near the bottom. The plate is typically a flat, single-flight helix. Rotation of the screw anchor causes it to pull itself into the soil. Several blades may be spaced along the shaft for greater capacity. Screw anchors have been used extensively on land as guy anchors. They have also been used to anchor offshore pipelines. Present offshore systems are installed in water depths to 300 m and have a pullout capacity of about 110 kN (25×10^3 lbs).

Relatively minor modifications are required to increase capacity by a factor of 2 (Raecke, 1973). However, scaling up to the sizes required for OTEC would be a monumental task. Used as the primary anchor, a screw-in anchor would be limited by lateral load capacity. As discussed in the section on piles, the sizes required to resist lateral loads are probably unreasonable. For this reason screw-in anchors will be considered only as supplementary anchors to assist in resisting vertical load.

Proposed Use. Single helix anchors might be used to supplement the vertical holding capacity and resistance to overturning of a deadweight anchor (Figure 16). The screw anchor and associated embedment system would be fabricated as an integral part of the deadweight. The deadweight

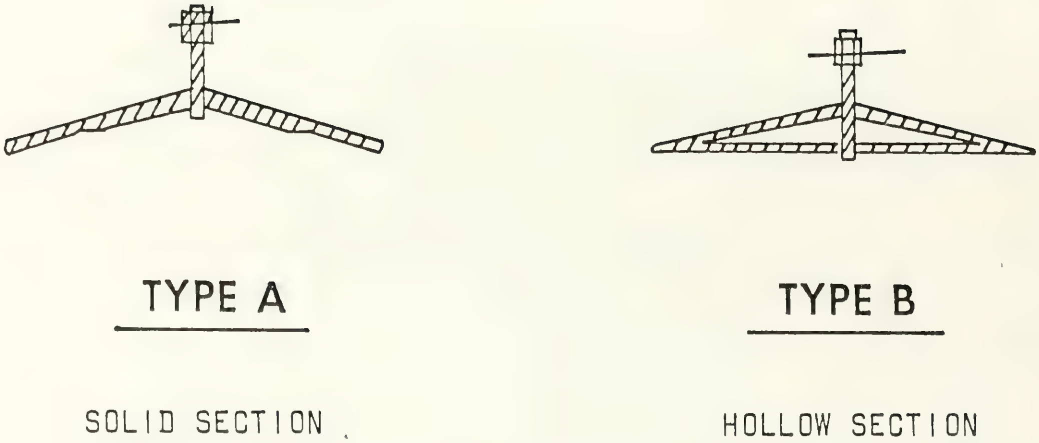


Figure 14. Cross-section of solid and hollow plate anchor design.

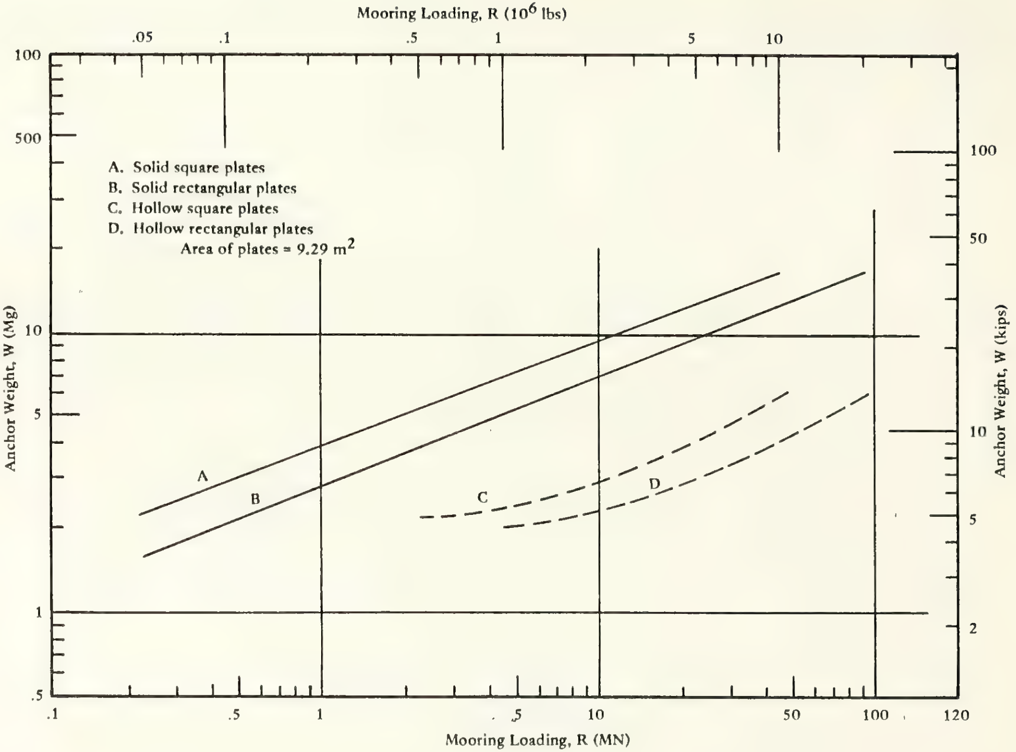


Figure 15. Required anchor weight versus mooring load for solid and hollow anchors.

able 10. Materials and Fabrication Cost Estimate for a Typical OTEC Plate Anchor

Environment - Deep Ocean

Soil category	A (soft clay)
Water depth	6000 m
Total mooring force	18 MN (4x10^6 lbs)

Anchor Description - nine, 3 m square plate anchors

Nominal holding capacity (each) - 3 MN (0.67x10^6 lbs)

Dimension	3 m x 3 m x 83 mm
Weight	5900 kg (13,000 lbs)
Efficiency	51.

Cost

Hy - 80 steel	\$0.77/kg
Fabrication (Welding)	3.63/kg
Unit Cost	4.40/kg
Sub-total (1 anchor)	26,000
Total (9 anchors required)	\$234,000

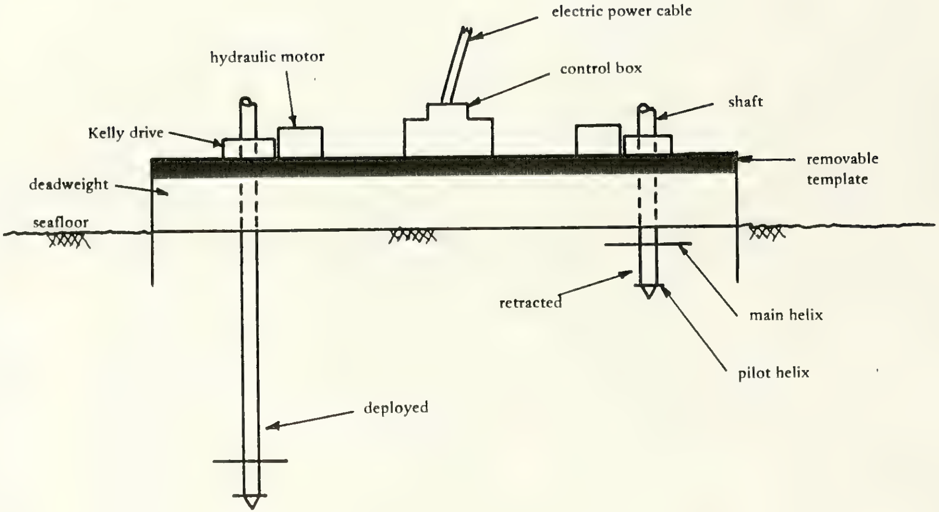


Figure 16. Composite deadweight and screw-pile anchor, resists high uplift loads.

and screw anchor combination would then be transported and positioned as a single unit. The helix would be embedded by hydraulic motors driven by electric power from the surface as proposed by Raecke (1973).

Only a single helix arrangement is considered feasible for the procedure suggested above. The added capacity of a multiple helix is outweighed by the complexities of installing it through the deadweight.

Holding Capacity. The holding capacity of a single helix anchor can be computed using procedures identical to the plate anchor procedures presented above. Approximate helix areas required may be found using Figures 7, 8, and 9. The representative plate anchor sizes in Table 9 may also be taken as representative helix sizes.

Embedment. The power required to embed helices of the sizes suggested by Table 9 is a matter of conjecture. The screw-in anchor concept proposed by Raecke required a 45 kW (60 hp) power source at the surface to install a single-helix anchor, 0.8 m diameter, to a depth of 15 m. Present offshore systems (Short, 1971) are powered by hydraulic motors requiring about 56 kW (75 hp). On the basis of fluke area, or holding capacity, an extrapolation of roughly 20:1 would be required for OTEC helix sizes. Power requirements are probably within reach.

Summary.

1. Single helix anchors, used to supplement the vertical capacity of a deadweight, are feasible. Further investigation should provide information regarding the economic aspects of the suggested concept.
2. Single or multiple helix anchors are not feasible as the primary OTEC anchor due to low lateral load-capacity.
3. Multiple-helix anchors do not appear suitable for use with deadweight because of installation complexities.

PILE ANCHORS

The evaluation of piles as OTEC anchors first considered a single pile, unrestrained head, subjected to a slowly applied horizontal and vertical load (Figure 17). Lateral and axial (pullout) capacities were determined for soil categories A, B, C, and D. Behavior of piles subjected to repetitive loading was estimated using available guidelines.

Pile Lateral Capacity

The effect of soil strength, pile length, and pile diameter on lateral capacity was determined by combining the results of a short and a long pile analysis. Overlap of the two procedures at a transition length (10 x diameter) allowed some comparison of results. Comparisons for piles of a given diameter were made for similar pile stiffnesses.

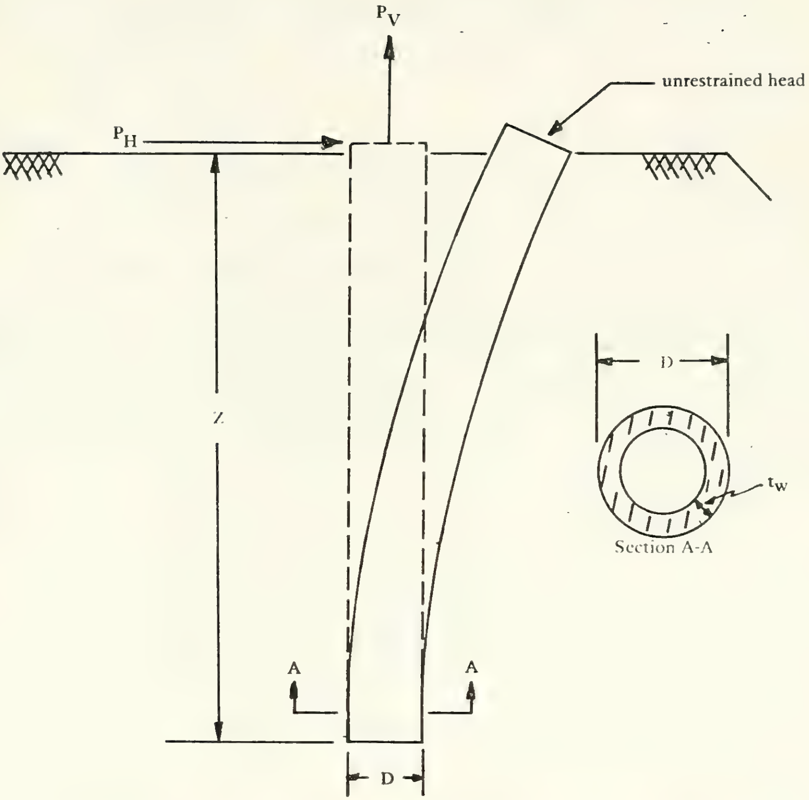


Figure 17. Deflection of a long pile by combined axial and lateral load applied at seafloor surface.

Short Pile Analysis. Czerniak's (1957) procedure for analyzing short, rigid piles was used. The procedure is based on the principle of static equilibrium. The pile is assumed to be infinitely rigid. It rotates about a point below the soil surface when an external lateral force is applied at the top of the pile (Figure 18). Soil resistance at each point along the pile is assumed to be proportional to pile deflection at that point.

$$p = Ky/D \tag{3}$$

where p = soil pressure (Pa)

K = modulus of horizontal subgrade reaction (Pa)

y = lateral deflection (m)

D = outside diameter of the pile (m)

Assuming a linear increase of unit soil resistance with depth, the length of pile necessary to maintain equilibrium under an applied load can be calculated.

For a horizontal load R_L, and moment Mo, applied at the surface, the appropriate equation of equilibrium is:

$$Z^3 - 14.14 \frac{R_L Z}{Dn_h} - 18.85 \frac{Mo}{Dn_h} = 0 \tag{4}$$

where Z = pile length (m)

R_L = lateral force at the surface (n)

n_h = coefficient of horizontal subgrade modulus (Pa/m)

Mo = moment at the surface (N.m)

for the special case of no external moment, equation (4) becomes:

$$R_L = \frac{Z^2 Dn_h}{14.14} \tag{5}$$

Similarly, the equation for bending moment at any point is:

$$M_x = R_L\, Z \,\{(\frac{x}{Z}) - 3\,(\frac{x}{Z})^3 + 2\,(\frac{x}{Z})\}^4 \tag{6}$$

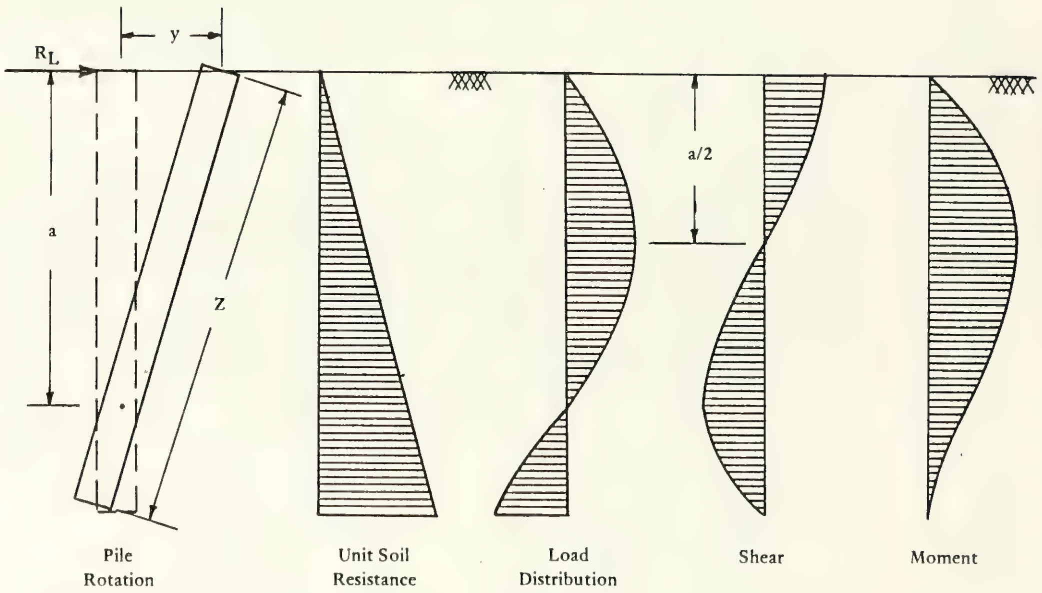


Figure 18. Short pile loading and displacement.

where x = distance along pile below the surface (m)

M_x = moment at point x (N·m)

For R_L applied at the surface, the maximum bending moment occurs at approximately 0.42Z. Substituting this for x in equation (6), the maximum bending moment becomes:

$$M_{max} = 0.260 \, R_L Z \qquad (7)$$

Equation (5) was used to find the lateral load capacity for pile diameters of 1.2, 2.4, 4.9, and 7.6 m (4, 8, 16, and 25 ft) for pile lengths up to the limit of validity of the Czerniak relationship, i.e., 10 x diameter. The coefficient of horizontal subgrade modulus, n_h, was selected based on the recommendations in Czerniak (1957) and Anon (1967). Those values for Soil Categories A, B, and C were based on the developed shear strength profiles, and the value for Soil Category D based exclusively on Anon (1976). The assumed values are listed in Table 11. Using these n_h values in equation (5) the lateral load versus pile length relationships shown as solid lines in Figures 19, 20, 21, and 22 were developed. Soil pressures were assumed to be sufficient to maintain equilibrium and resulting deflections were assumed to be acceptable.

Long Pile Analysis. A long pile analysis was done for two reasons:

(1) To find out if pile capacity could be increased using longer piles.

(2) To check the short pile results at the transition length (Z = 10D).

A subgrade reaction method (Gill and Demars, 1970) was used to find lateral capacity for selected pile lengths and diameters. The method is similar to a commonly used method developed by Matlock and Reese (1960). The non-linear soil load-deformation characteristics (p-y curves) are represented by a rectangular hyperbola. Hyperbola shape is defined by measurement of the undrained shear strength for clay; and effective overburden and friction angle for sand. The procedure employs a finite difference solution of the equation of bending to obtain deflections. Soil properties, pile characteristics, and pile loading are input to the computer program. Pile stiffnesses were selected on the basis of maximum bending moments calculated from the Czerniak rigid-pile analysis for Soil Category C. The pile sections required to obtain these stiffnesses appear reasonable and practical for fabrication. The long-pile analysis program returns pile deflection, bending moment and soil modulus as a function of depth. The external loading boundary condition was adjusted until pile failure (bending moment exceeded) occurred.

Table 11. Assumed Coefficients of Horizontal Subgrade Modulus

Soil Category	n_h	
	(kPa/m)	(psf/ft)
A (Clay)	14.5	92
B (Clay)	21.8	139
C (Clay)	30.8	196
D (Sand)	84.8	540

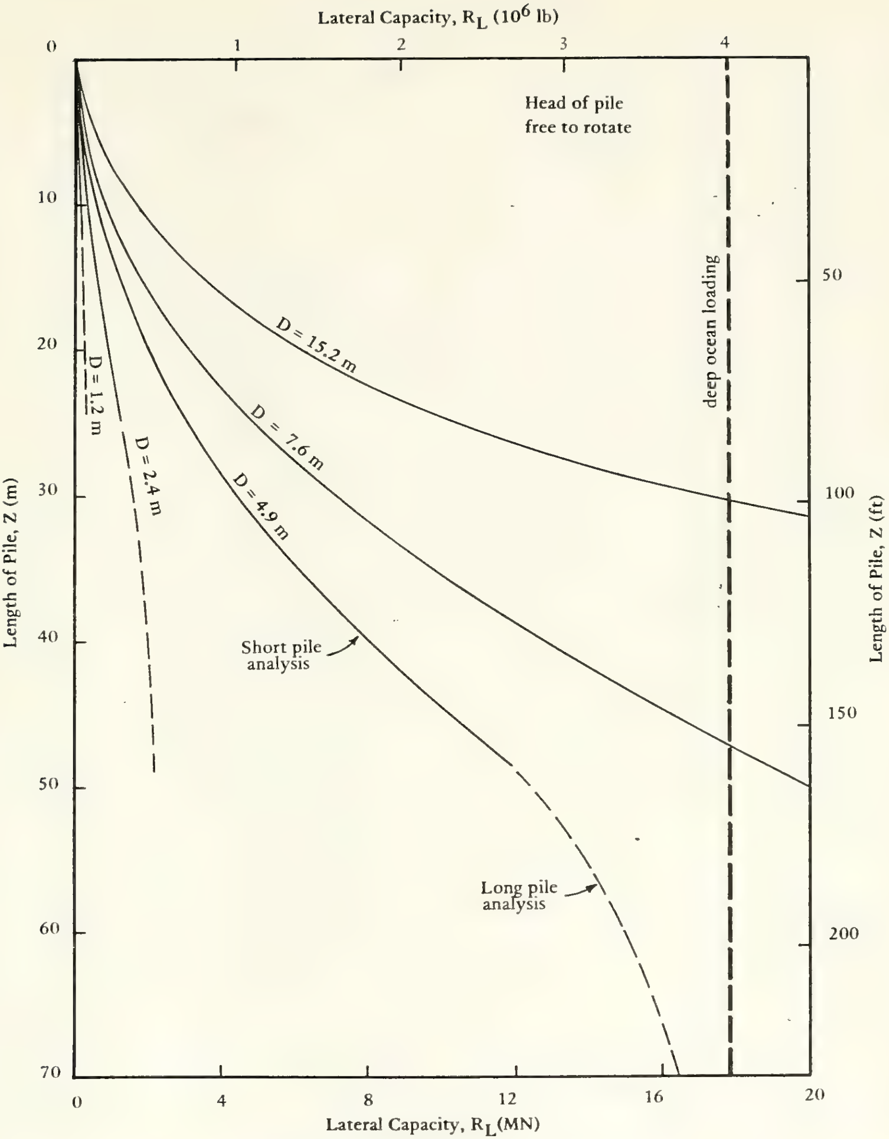


Figure 19. Pile lateral capacity in soil category A.

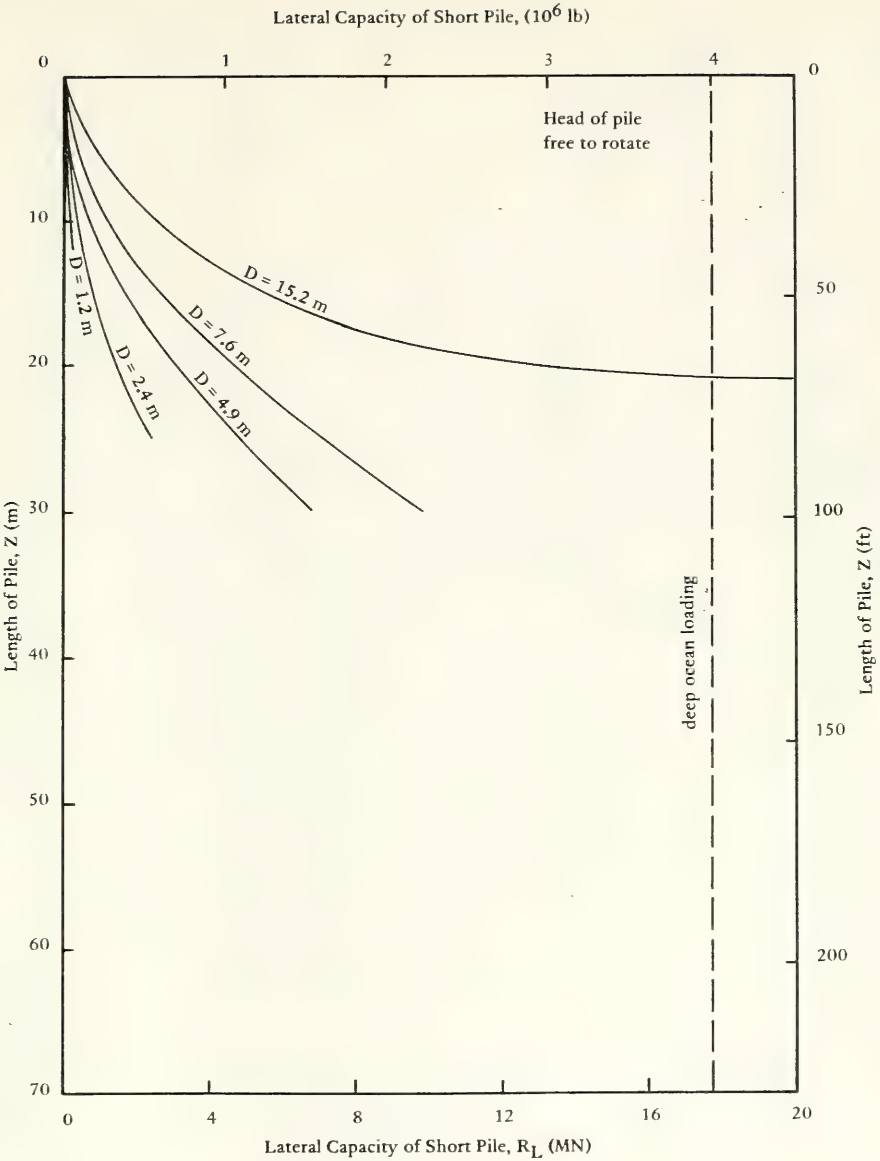


Figure 20. Pile lateral capacity in soil category B.

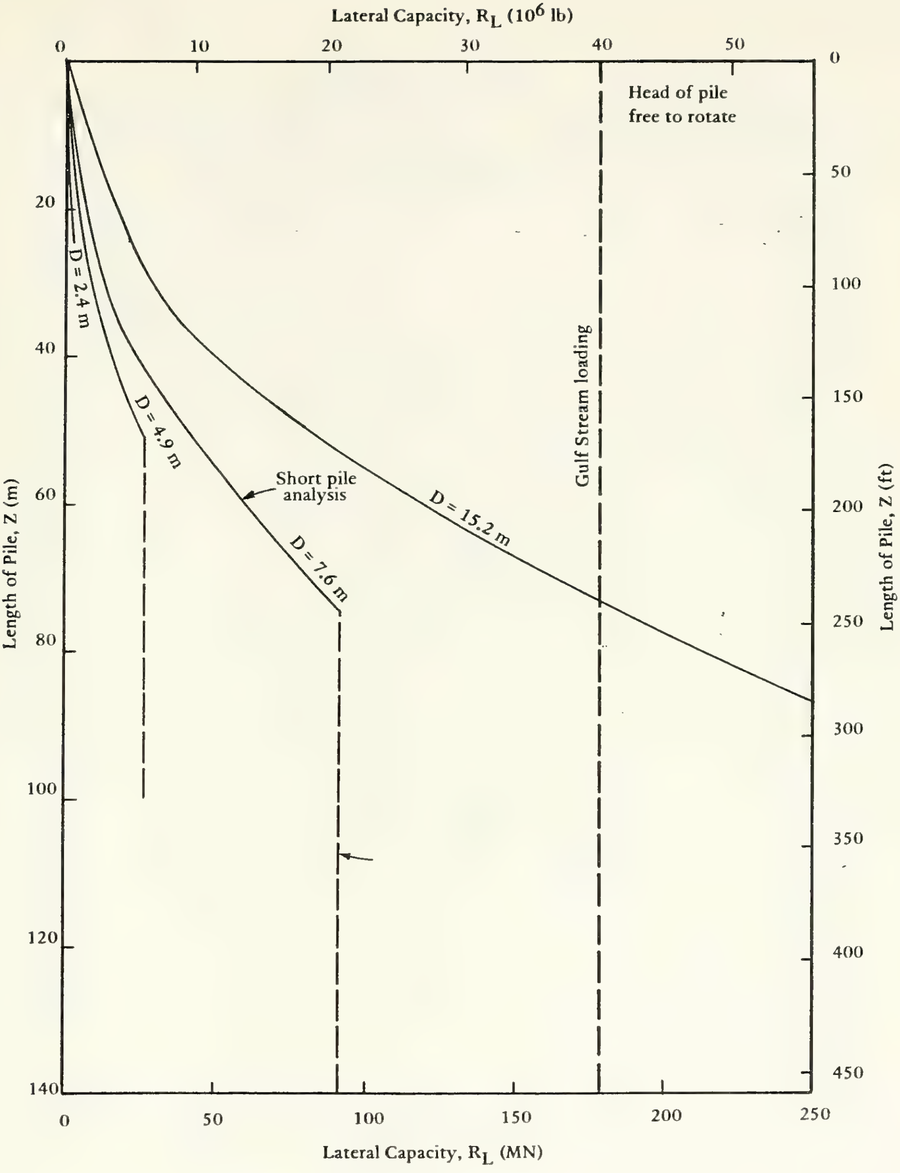


Figure 21. Pile lateral capacity in soil category C.

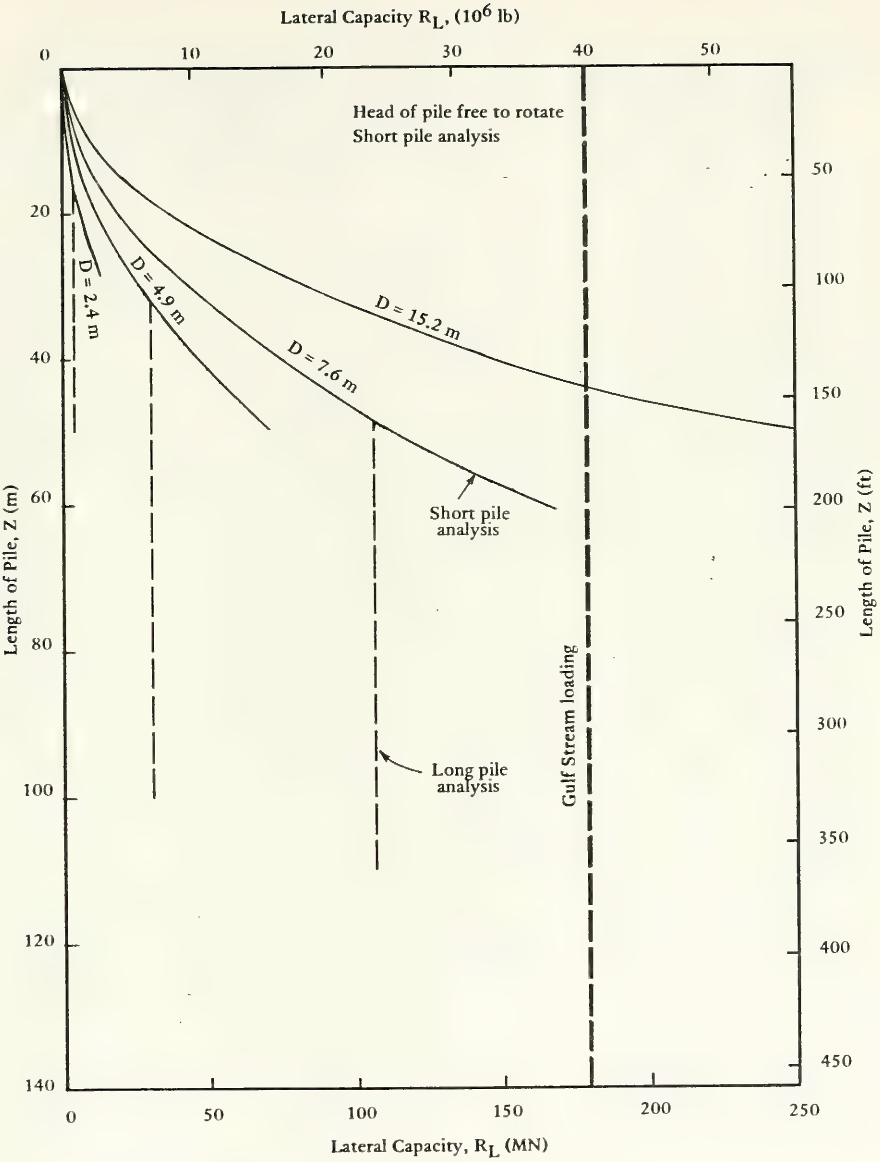


Figure 22. Pile lateral capacity in soil category D.

Long pile analysis lateral load capacities for various lengths and diameters are shown as dashed curves in Figures 19, 21, and 22. A long pile analysis was not done for category B soil (Figure 21). Curves were drawn through the more conservative capacity at the transition length between the long and short pile analysis, i.e., at Z = 10 x D. The solid and dashed lines delineate the lateral capacity envelope for the piles shown.

Results. Review of Figures 19 and 20, indicative of pile anchor performance in deep ocean environments, suggests that single piles of 7.6 m diameter (25 ft) and 47 m in length would be required to mobilize the necessary 18 MN ($4x10^6$ lbs) lateral load resistance.

Figures 21 and 22, indicative of performance in a competent clay and sand seafloor, as might be found beneath a high energy area, suggests that a single pile anchor design is probably not appropriate for such an area. Piles of 7.6 m diameter (and of the stiffnesses assumed) are not capable of resisting the full 180 MN horizontal load component ($40x10^6$ lbs). Larger diameter piles could mobilize sufficient soil resistance to resist the Gulf Stream loading, however, technical feasibility suggests that a multiple pile anchorage using smaller piles would be more reasonable.

Figure 23 illustrates the influence of the pile analysis procedure employed. The short pile analysis assumed mobilization of the available soil resistance and maximum bending moment in the short pile was calculated. Then a pile section was designed to resist that moment. This pile section was then input into the long pile analysis (or more properly the stiffness values were input). This fixing of the pile stiffnesses results in the fixing of the bending resistance of the respective pile diameters. For lengths greater than about 10 diameters; lateral load capacity is controlled by the assumed bending resistance. Adherence to this procedure is justified because the pile secitons designed appear practical as is: increasing the pile stiffnesses would result in very thick-walled, unreasonable, possibly unfabricatable sections. The actual pile sections designed will be presented and discussed later; it is important to note here that the break in the lateral load capacity curve (Figure 23) is not indicative of a soil phenomenon, but rather is due to the pile stiffness assumed in the analysis. The data presentation of Figure 23 suggests that for the pile sections used, there is no marked improvement in lateral load capacity when the pile is lengthened beyond 10 diameters. However, increasing the length beyond 10 diameters may decrease pile deflection. Figure 24 shows deflection at maximum load for the 8 ft. diameter pile at Z = 10D and Z = 20D. The figure indicates much larger deflection at the shorter length. At Z = 10D, the pile is definitely exhibiting the short pile behavior idealized in Figure 18. Increasing length to 20 diameters gives the smaller deflections characteristic of long pile behavior (Figure 17). At 30 diameters length deflections would be almost the same as the 20 diameter case. The point at which increases in length do not give a corresponding decrease in deflection depends on a well-known stiffness factor (Gill and Demars, 1970, and others):

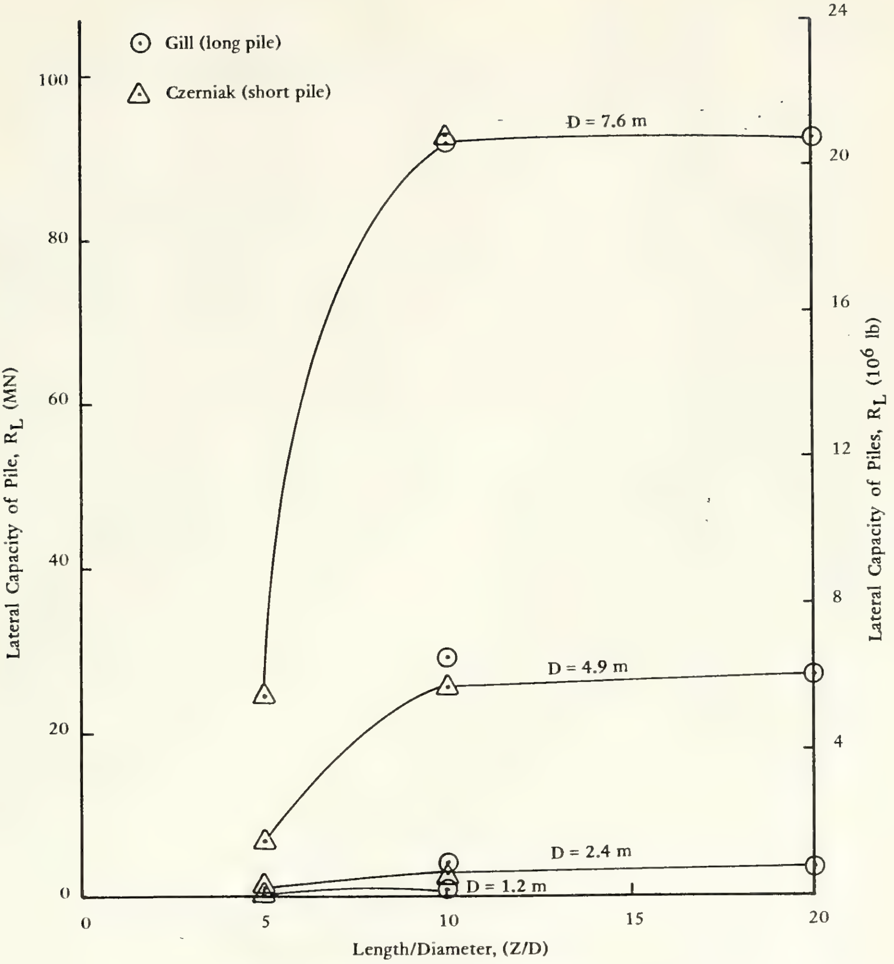


Figure 23. Pile lateral capacity versus normalized length.

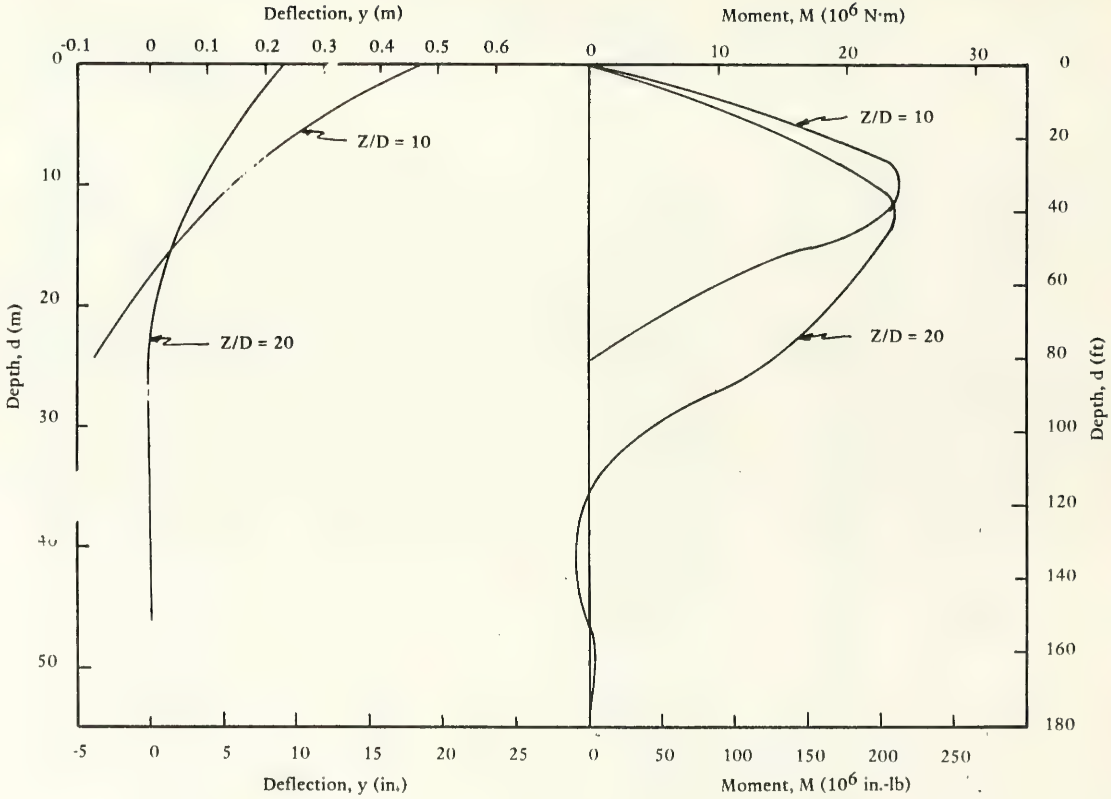


Figure 24. Pile deflection and moment versus soil depth (at failure loading) for the 2.4m diameter pile.

$$T = \left(\frac{EI}{n_h}\right)^{1/5} \qquad (8)$$

where T = relative stiffness (m)

E = Young's modulus for the pile (Pa)

I = polar moment of inertia of the pile (m^4)

n_h = coefficient of horizontal subgrade modulus (Pa/m)

Piles with embedded lengths larger than 4T act as if they were embedded to an infinite depth (Davisson and Gill, 1963).

The desirability of small deflection takes on increased significance when repetitive loading is present. This will be discussed more in the section on repetitive loading.

Pile Axial Capacity

Pullout Capacity in Clay. A semi-empirical procedure (Vijayvergiya and Focht, 1972) was used to predict the pullout capacity of the piles for cohesive soils (A, B, C). The method of analysis expresses pile fricitonal resistance as a function of mean vertical effective stress and mean undrained shear strength:

$$Q_s = \lambda\, (\bar{\sigma}_m + 2s_m)\, A_s \qquad (9)$$

where Q_s = side friciton on pile (n)

λ = dimensionless friction coefficient

$\bar{\sigma}_m$ = mean vertical effective stress over pile length (Pa)

A_s = lateral area of embedded pile (m^2)

s_m = mean undrained shear strength over length of pile (Pa)

The friction coefficient was based on available data from load tests at a number of different locations. Comparison of predicted friction pile resistance to observations showed good agreement (Vijayvergiya and Focht, 1972).

Pullout Capacity in Sand. For category D soil (sands) a simplified empirical procedure was followed. Pullout resistance was taken as a function of mean vertical effective stress, $\bar{\sigma}_m$:

$$Q_s = K_s A_s \overline{\sigma}_m \tan \delta \qquad (10)$$

where K_s = coefficient of lateral pressure on pile wall

δ = effective friction angle of soil on pile wall (rad).

For the calcareous sand, δ was taken as 0.44 rad (25 deg) and K_s as 0.4. Such high values are appropriate for calcareous materials only when the piles are installed by drilling and grouting (McClelland, 1974). Available large scale test information (Angemeer et al., 1973, 1975) indicates that capacities of driven piles in calcareous sands may be as little as one-fourth of those indicated by Equation 10. Pile load tests are required to accurately predict the capacity of driven piles on this type material. The size and difficulty of installation of a pile anchor for the OTEC power plant dictates that the soil resistance available be maximized. Therefore, this study assumes that the technology will be developed to provide the drilled and grouted pile option in water depths to 6,000 m. In reality, the installation of large anchor piles in deep water appears more technically feasible by the drilling and grouting technique than by the driving technique.

Results. Computed pullout capacities are shown in Figures 25, 26, 27, and 28 for respective soil categories. The reader should note that capacities were calculated independently of lateral load. Again, calculations were for a single, unrestrained pile subjected to a slowly applied vertical load. Note that a 110 m long, 4.9 m diameter pile would provide sufficient pullout resistance in the deep ocean environment. An 85 m long, 7.6 m diameter pile would hold in the Gulf Stream environment. These pile lengths and diameters are probably within present offshore capabilities.

Effect of Repetitive Loading

Definition. Repetitive loading is defined as successive, slow applications and relaxations of a load. In other words, shock loads are excluded. Repetitive load is a reasonable approximation of what an actual OTEC anchor will encounter.

Discussion. Several authors (Matlock, 1970; Gill and Demars, 1970) have noted that static load displacements may be increased by as much as 50 percent under repetitive loading. At initial displacements larger than 20 percent of pile diameter, Matlock (1970) noted a continuous and progressive deterioration of soil resistance with cycling. Figure 24, presenting displacement data for a 2.4 m diameter pile, indicates the 24 m long pile will deflect 0.5 m under the static loading, or about 21 percent of the pile diameter. The longer, 48 m pile will deflect only 0.26 m or about 11 percent of the diameter. If this relationship holds for piles of other diameters and in other soils, then piles longer than 10 diameters (or, alternatively, reductions in the lateral loads applied to the piles) will be necessary to prevent progressive deterioration of soil resistance.

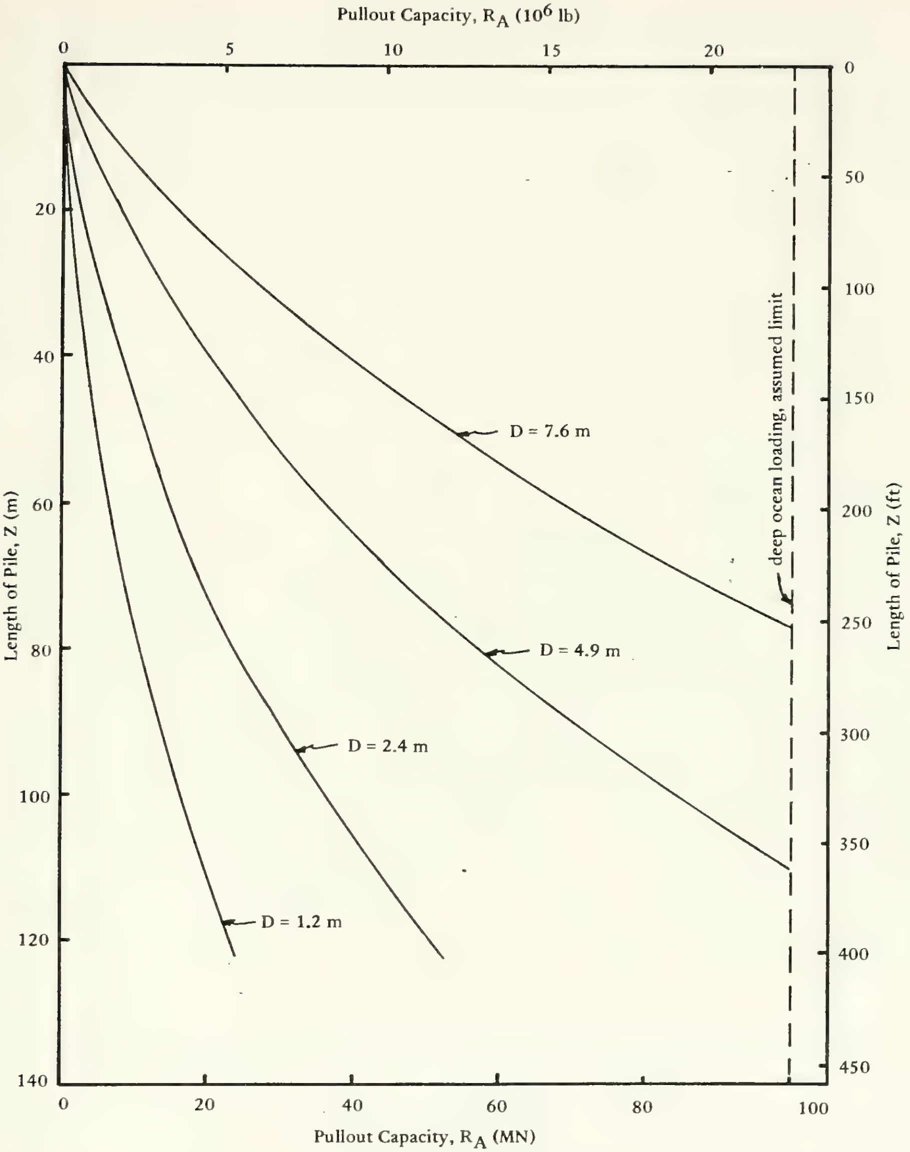


Figure 25. Pile axial pullout capacity in soil category A.

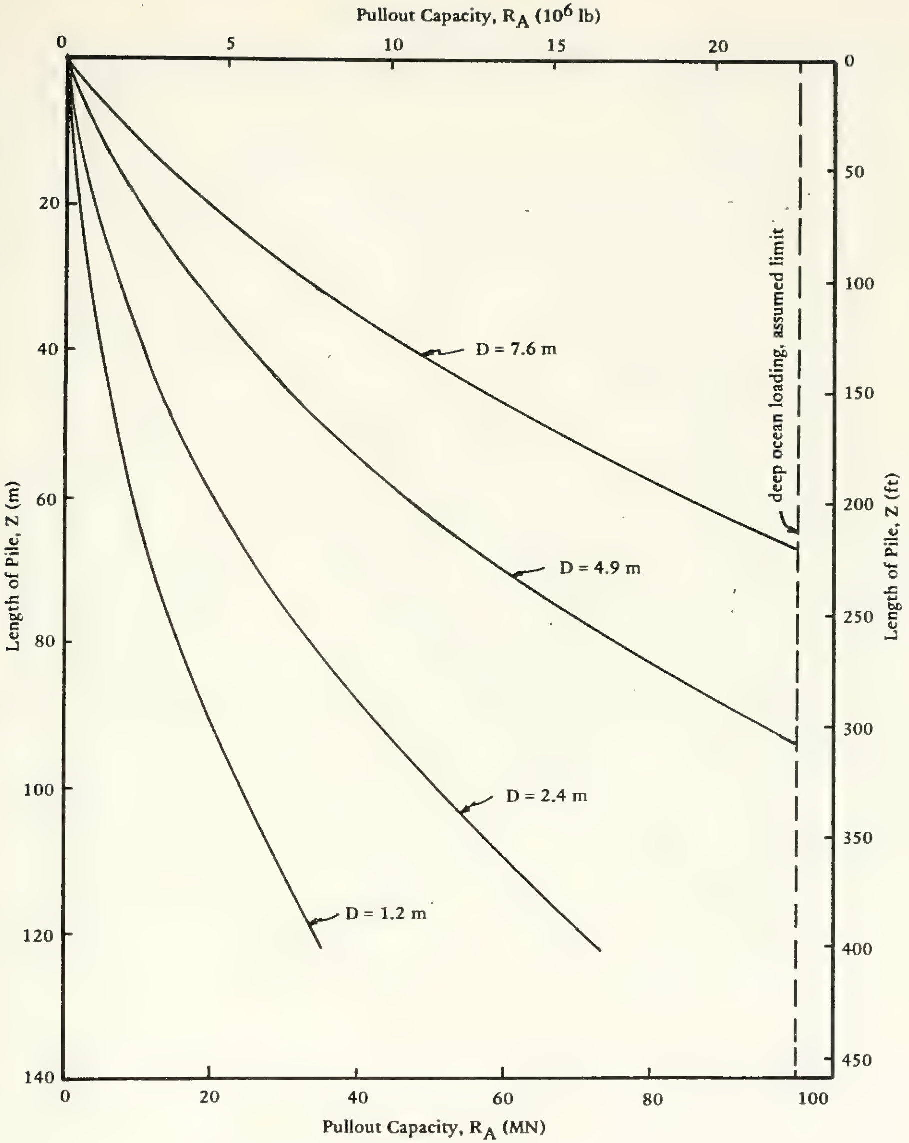


Figure 26. Pile axial pullout capacity in soil category B.

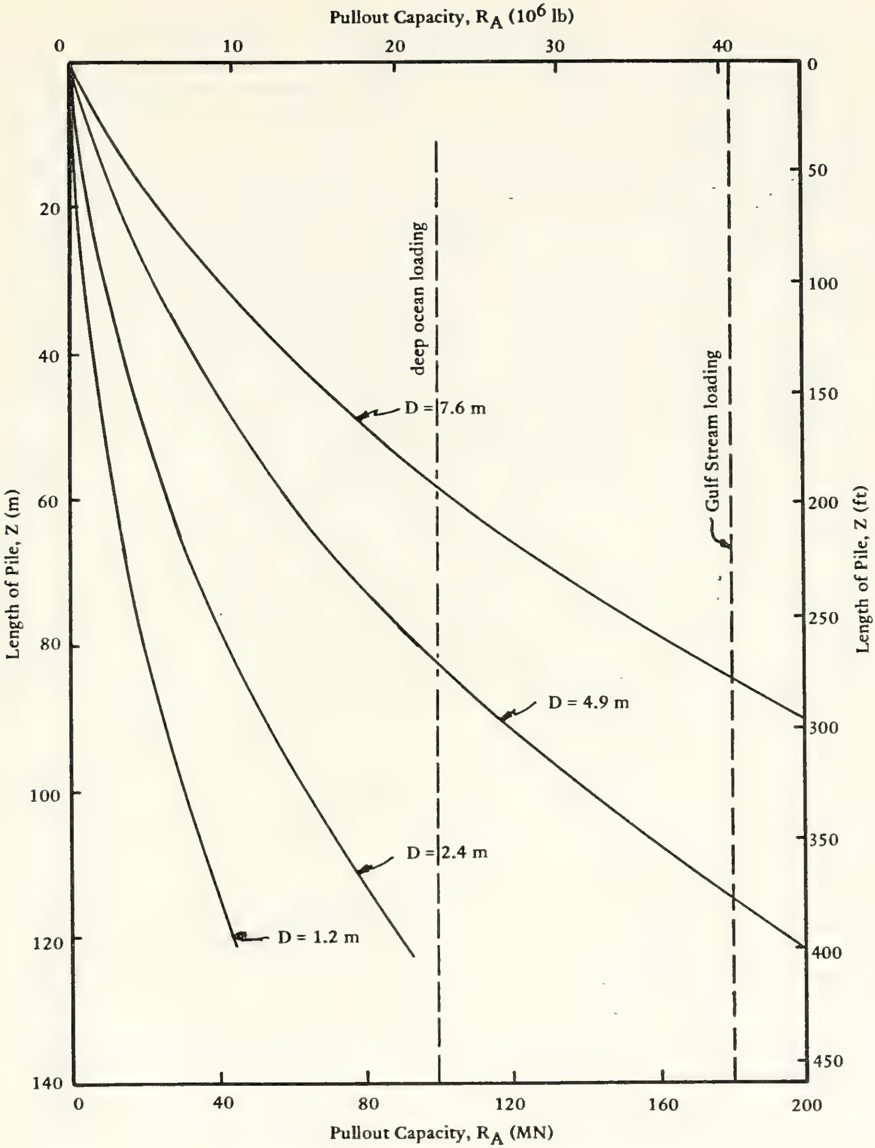


Figure 27. Pile axial pullout capacity in soil category C.

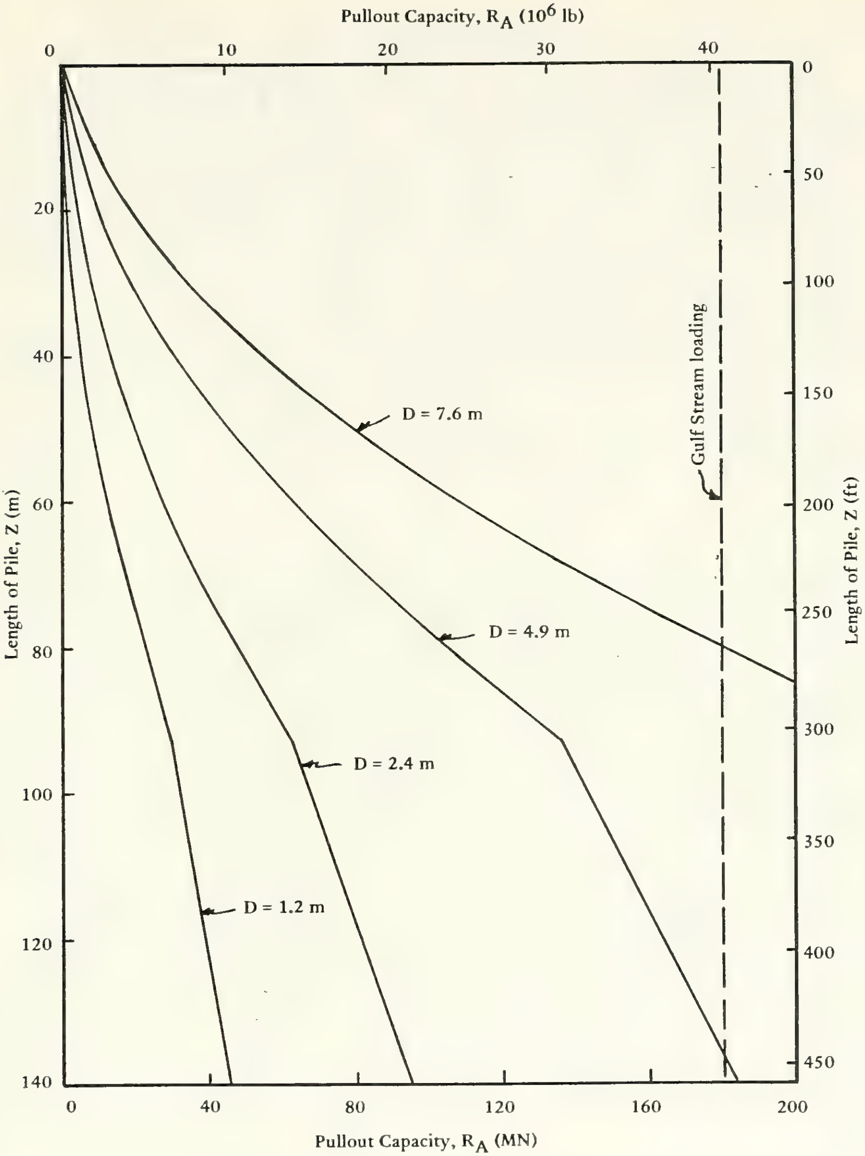


Figure 28. Pile axial pullout capacity in soil category D.

Several authors reported bending moment increases due to repetitive loading. Davisson and Salley (1968) noted a 20 to 50 percent increase in bending moments on 1.2 m diameter drilled piers. In model and field tests, Matlock (1970) observed moment increases of almost 100 percent during cyclic loading. Several of Matlock's tests were performed on a 0.33 m diameter instrumented pile in soft, slightly over-consolidated clay. These conditions are fairly representative of marine sediment deposits. Matlock suggested that a reduction in pile lateral capacity due to cycling might be estimated on the basis of a ratio of pile deflection to diameter. The rough guidelines in Table 12 were estimated from the results of his model tests.

A typical deflection ratio expected for the OTEC anchor is about 0.1. Thus, design capacity for repetitive loading is about 0.58 times static capacity. Please note these reductions have not been applied to the data presented herein.

The effect of repetitive axial loading on axial pullout capacity should be minimal (Bea, 1975). Likewise the effect of repetitive lateral load on axial capacity has not been a serious problem in present offshore structures.

Preliminary Structural Design of Required Piles

As indicated in the lateral load capacity analysis section, the pile sections were designed to resist the maximum load conditions for piles with lengths equal to 10 diameters in the category C soil. This course of action was taken early in the study to enable an evaluation of the materials available for fabrication, the wall thicknesses and weights necessary, and the overall practicality of the pile anchor concept applied to OTEC. Table 13 presents a summary of results for two, potential, single-pile anchor designs, one a 4.9 m diameter, 31 m long pile, the other a 7.6 m diameter, 7.6 m long pile. These piles in a category C soil are capable of mobilizing soil resistance to supply the noted axial and lateral load capacities, with no reductions applied for repetitive loading. In all cases, the large bending moments resulting from the high lateral loads controlled structural design of the piles.

Reinforced concrete was eliminated from consideration because the high bending moment combined with axial tension would result in significant cracking on the tension side of the pile, an unacceptable condition in the marine environment. Either prestressed concrete or steel could be used for the 4.9 m diameter pile resisting the 9.8 MN (2.2×10^6 lbs) lateral load. The prestressed concrete pile would, however, require 13.3 MN (3×10^6 lbs) of steel and concrete to resist the 9.8 MN lateral load. Thus, the concrete pile anchor efficiency (lateral load capacity to weight ratio) would be 0.7--little better than a good deadweight anchor. The steel pile appears much more desirable with a 50 mm wall thickness and a lateral load efficiency of 5.4.

An adequate design using prestressed concrete is not possible for the 7.6 m diameter pile resisting the 97 MN (22×10^6 lbs) lateral load. One simply cannot fit the required steel and concrete into the cross-section available. An idealized steel pile section is suggested for

TABLE 12. Suggested Residual Lateral Load Capacity Percentage Due to Repetitive Loading as a Function of First Cycle Deflection

y/D	Lateral Capacity (percent of static capacity)
0.2	20
0.1	58
0.05	62
0.025	67

Table 13. Pile Anchor Characteristics, Category C Soil

Diameter, m	4.9		7.6
Length, m	31		76
Environment of Possible Use	Deep Ocean		Gulf Stream
Axial Load, MN/x10^6 lbs	22.7/5.1		149/33.6
Lateral Load, MN/x10^6 lbs	9.8/2.2		98/22.0
Material	Prestressed concrete	Steel	Steel
Grade	f_c' = 41 MPa(6000 psi)	A-36	HY-80
Cross Section	Solid	Shell	Shell
Wall Thickness, mm	———	51	152
Maximum Stress, MPa/ksi	———	104/15	321/47
Number of 42.8 mm Diameter Strands	188	———	———
Total Weight, Mg/x10^6 lbs	1400/3.0	185/0.41	2100/4.7
Lateral Load/Weight	0.7	5.4	4.6

illustrative purposes. The 7.6 m diameter pile would require pile walls 150 mm (6 in.) thick. HY-80 steel can be obtained in such thickness, is weldable, and would serve well in seawater at the ambient $4^{o}C$; it is not known if such plates can be formed to the necessary shell curvature. Of course, the real pile structure would probably look considerably different from the idealized simple shell. One fact is known, the pile strucutre will be steel and will not be reinforced or prestressed concrete.

Summary

Probable anchor pile ultimate capacities are shown in Figure 29 for the deep ocean environment and in Figure 30 for the Gulf Stream environment. In reviewing and using this ultimate capacity data, the reader must remember that safety factors on the order of 2 must be applied to this data when determining the safe working load. Working loadings higher than this will cause excessive pile deflections leading to pullout or failure in bending. Capacities for axial and lateral load were computed independently. Pile lengths plotted in Figure 29 were chosen to: (1) minimize deflection and (2) reflect practical constraints (installation, fabrication, anticipated costs). Figure 29 suggests that single piles used to anchor OTEC power plants in the deep ocean will have to be about 4.9 m in diameter to resist the lateral load component. The pile length required would vary with the vertical load component, from about 50 m long for zero vertical load to about 120 m long for a 100 MN vertical load component (see Figure 25 for source of pile length data).

Figure 30 indicates the difficulty of obtaining the very high lateral load capacities necessary for the Gulf Stream environment.

Pile anchors of conventional diameter, say 2.4 m, linked in groups, do not appear economical for resisting the large lateral forces encountered in locations like the Gulf Stream. To resist repetitive lateral load with piles alone would require as many as 118 2.4 m diameter, 49 m long piles.

In the less severe deep ocean environment piles could prove to be effective and economical. They have one distinct advantage over deadweights. Pile lateral capacity is not severely reduced as axial pullout forces increase. Thus piles are better able to cope with high mooring line angles than are deadweight or drag embedment anchor systems.

DEADWEIGHT ANCHORS

Introduction

Horizontal Load Resistance. Deadweight anchors derive their lateral load capacity by developing friction between their bottom surface and the seafloor (Figure 31). The maximum horizontal resistance, R_L, between the seafloor and the deadweight is a function of the gravitational force imposed by the deadweight on the seafloor and of the coefficient of friction between the deadweight and the seafloor. The coefficient of friction varies widely with seafloor material type from 0.1 to 0.5 with the type of seafloor material and the nature of the deadweight anchor

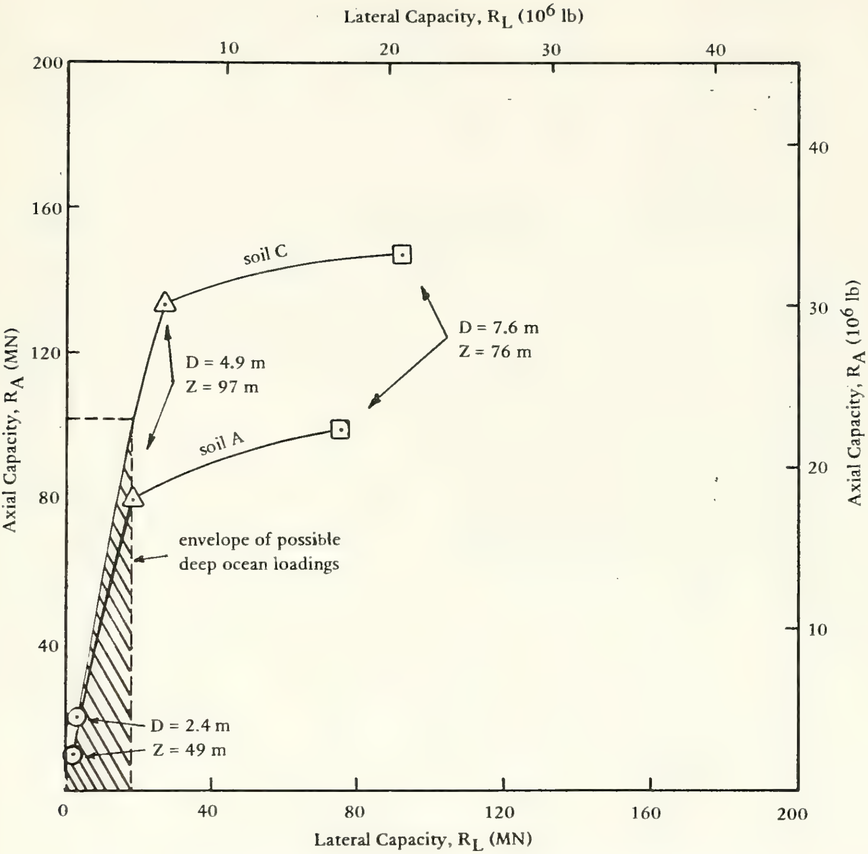


Figure 29. Pile anchor holding capacity under static load, deep ocean environment.

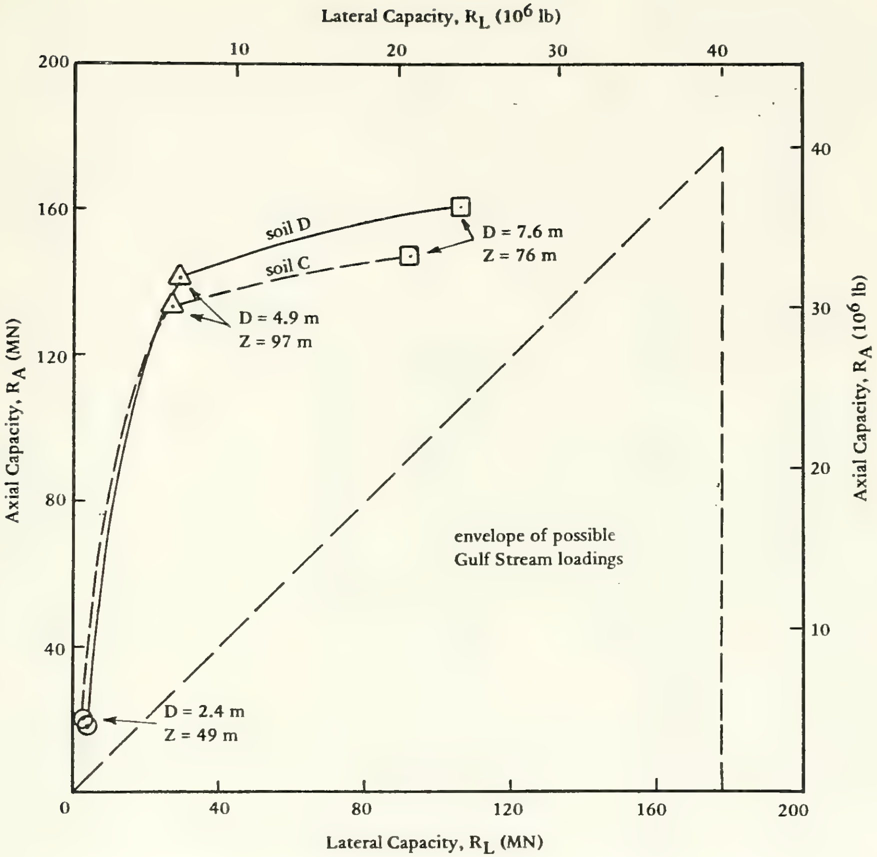
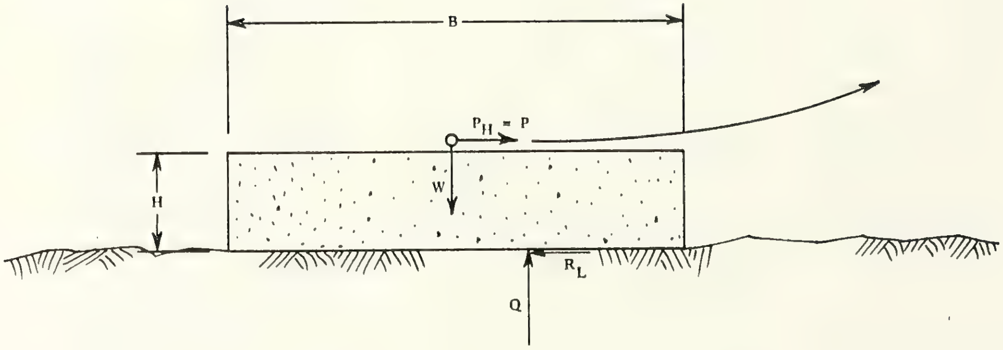


Figure 30. Pile anchor holding capacity under static load, Gulf Stream environment.



$$(P_H)_{max} = \mu W$$

Figure 31. Simple deadweight anchor with zero line angle, i.e., mooring line horizontal at the anchor.

bearing surface. For the most part, on the predominantly soft cohesive seafloor sediments, typical values are 0.1 to 0.2.

One purpose of deadweight anchor design is to increase the lateral holding capacity by providing roughened surfaces or teeth to better engage the seafloor surface. Cutting edges, often referred to as skirts in the case of cutting edges around the perimeter of a deadweight, are a common solution (Figure 32). When the cutting edges have been embedded vertically in the seafloor, lateral sliding of the deadweight occurs on shearing zones (possibly planes) emanating from the tip of the cutting edges. Thus the cutting edges serve to force the shear zone into a typically stronger soil. In addition, for the deadweight to move horizontally, the wedge of soil in front of the leading skirt must be pushed up and out of the way providing another component of lateral load resistance.

Vertical Load Resistance. Increasing the mooring line angle at the anchor from zero causes a vertical component of load to be applied to the anchor. This vertical load component must be resisted by a portion of the submerged weight of the deadweight anchor. (Figure 33).

Overturning Resistance. The deadweight anchor must have sufficient resistance to overturning arising from its shape, distribution of mass, and location of mooring line attachment to remain stable under any loading condition. The deadweight must be very flat or squat, that is, it must have a large width, B, as compared to its height, H. The center of mass must be located as low as possible. Lastly, the mooring line attachment point on the deadweight must be positioned as low on the anchor as possible so as to minimize the lever arm of the horizontal load component, P_H. To minimize overturning potential, much of this study assumes the deadweight anchor height to be one-tenth of the width or

$$H = 0.1B \tag{11}$$

The cutting edge length Z, also influences the overturning potential. Deadweights with low ratios of cutting edge penetration to anchor width, i.e., Z/B, will have a lesser potential for overturning.

Deadweight anchors required to resist equal loads from any direction offer little flexibility in varying the point of mooring line attachment. The attachment point must be centrally located allowing only movement to a point lower on the anchor profile. However, deadweight anchors which must resist their heavier loadings from one general direction offer potential for optimizing the attachment point location. Rather than elaborate here, the reader is referred to the Pearl Harbor concrete anchor illustrated in Figure 10(d) of CEL TM-42-76-1 (Valent et al., 1976). This report is limited to consideration of centrally loaded, square-plan deadweight anchors.

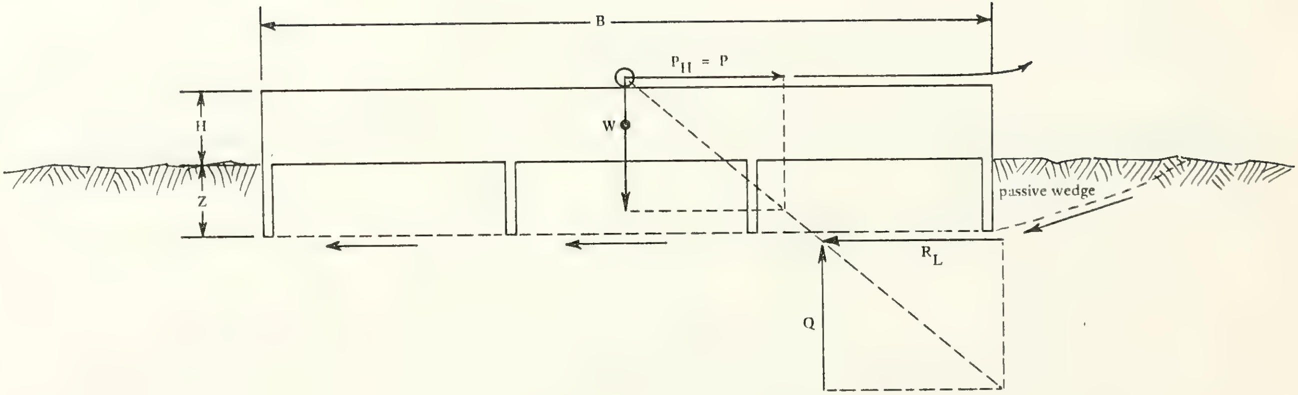


Figure 32. Deadweight anchor with cutting edges, zero line angle.

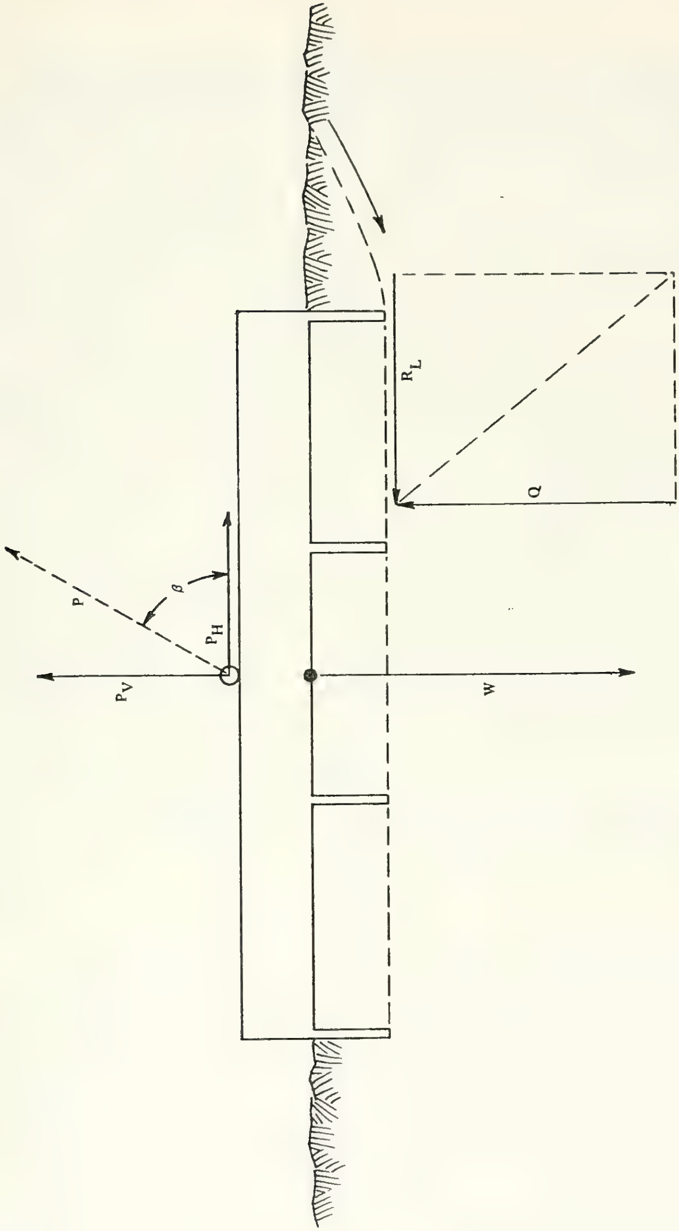


Figure 33. Deadweight anchor with cutting edge, nonzero line angle, β.

Definition of Deadweight Anchor Failure. This study assumes that a deadweight anchor has failed if: (1) the bearing capacity of the supporting sediment is exceeded leading to a local or general failure of the deadweight, (2) the lateral load capacity of the sediment is exceeded leading to excessive permanent lateral displacement of the deadweight, and (3) the total "immediate" and "consolidation" settlement is such that the mooring line attachment point to the deadweight is lowered to beneath the pre-existing seafloor surface. (This last mode of failure, that of settlement, has not been treated herein.) Items (1) and (2) separately and individually define any local bearing capacity failure, necessarily involving tilting and some lateral displacement of the deadweight, as a deadweight anchor failure.

Although the assumed definition of anchor failure does require that the deadweight anchor not slide under the ultimate loading condition, the authors allow that this condition may be too stringent. Under certain circumstances it would appear advantageous to design a deadweight anchor to allow some sliding as a mechanism for accommodating peak mooring line loads. Such a sliding system would require sufficient area for movement without adversely affecting the ecosystem or other users of the sea. This study has not treated such a sliding deadweight anchor system.

Approach. The sizing of deadweight anchors of the type required for OTEC is dependent primarily on the magnitude of the lateral load component. This study first evaluates the lateral load capacity as a funciton of deadweight anchor width, B, and cutting edge penetration, Z. Then the cutting edges are structurally designed to resist the lateral load applied to them with the prime objective of obtaining the cutting edge thicknesses, t, required. When cutting edge thicknesses are available, the load required to fully embed the skirts can be calculated. Further, the deadweight configuration can then be optimized to provide the minimum mass required to provide a given lateral load capacity.

Deadweight bearing capacity is treated subsequently to demonstrate that, for a deadweight anchor designed to resist the expected lateral loads for OTEC, bearing capacity failure is not an insurmountable problem.

Evaluation of Lateral Load Capacity

Failure Mode. The lateral load capacity of the deadweight anchors was evaluated assuming the sliding mode pictured in Figure 33. This failure mode is most desirable because the base shearing zone is forced down into the deepest possible position where the soil shearing strength is greatest for the strength profiles expected. Development of this failure mode is dependent on the ratio of cutting edge spacing to length, b/Z; on the net vertical load magnitude at maximum mooring line load; and on the applied overturning moment. The assumption here is that cutting edge spacing, net vertical load, and overturning moment will be adjusted to ensure development of the deep, planar failure zone.

The lateral load capacity was assumed to include resistance from a complete passive wedge failure in front of the leading cutting edge. The existence of a complete wedge assumes no scour about the anchor block.

This condition could be expected in most seafloor areas; however, in many category D and some category C environments scour could be a problem. Deadweight anchor design should seek to minimize the potential for scour by streamlining the deadweight and by providing scour protection about the anchor periphery.

In order to minimize the potential for overturning, initial considerations of lateral load capacity assumed a ratio of cutting edge length, Z, to deadweight width, B, of 0.1. Later evaluation of the lateral load developed as a funciton of the anchor deadweight required to embed the skirts showed the ratio of Z/B of 0.1 to also be very desirable from the standpoint of design efficiency.

Analysis of Passive Wedge Component.

1. Cohesive Soil. The analysis of the passive wedge component for a cohesive soil (categories A, B, and C) utilized the relationship

$$R_p^1 = 1/2\gamma Z^2 + 2sZ \qquad (12)$$

where R_p^1 = lateral load resistance per unit width of passive wedge (N/m)

γ = buoyand unit weight of soil (N/m^3)

Z = length of cutting edge (m)

s = undrained shear strength of soil (Pa)

Values of undrained shear strength of the seafloor sediment, s, and of the buoyant unit weight of the sediment, γ, used in the analysis are as presented in Figure 1 and Table 1. The total lateral resistance arising from the passive wedge is found by multiplying the lateral load resistance per unit width, R_p^1, by the width of the passive wedge. For the square plan deadweight anchors treated herein, the width of the passive wedge would be the block width, B. The passive wedge component of the lateral load resistance is then:

$$R_p = R_p^1 B \qquad (13)$$

This value of the passive wedge lateral load resistance is believed realizable at all mooring line directions, i.e., the same value can be realized even when the mooring line is acting along a diagonal of the square deadweight anchor block. When evaluating the lateral load capacity of circular plan deadweight anchors, it would appear reasonable to reduce the average passive pressure acting on a diameter as is done when evaluating the lateral load capacity of cylindrical piles (Czerniak, 1957). Lateral load capacities derived from the passive wedge are presented as a function of anchor side dimension for Soil Categories A and C in Figures 34 and 35 respectively.

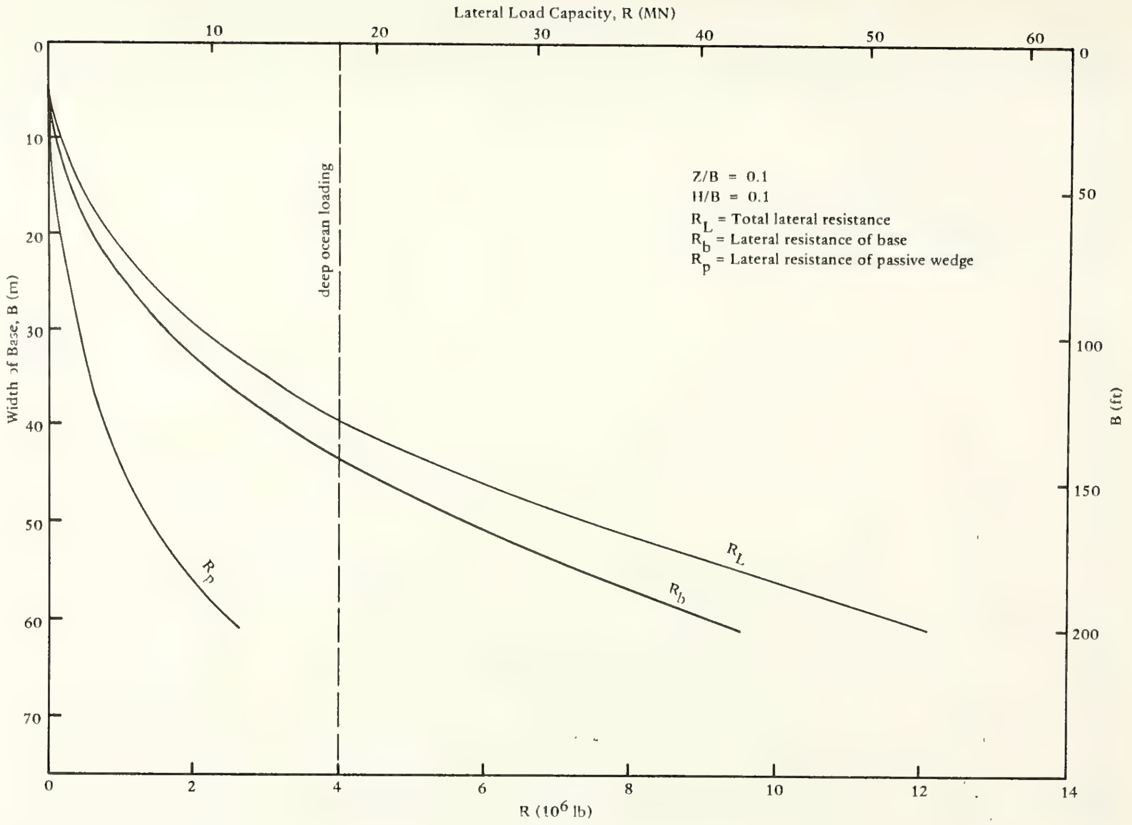


Figure 34. Lateral capacity of deadweight anchors versus width in soil category A.

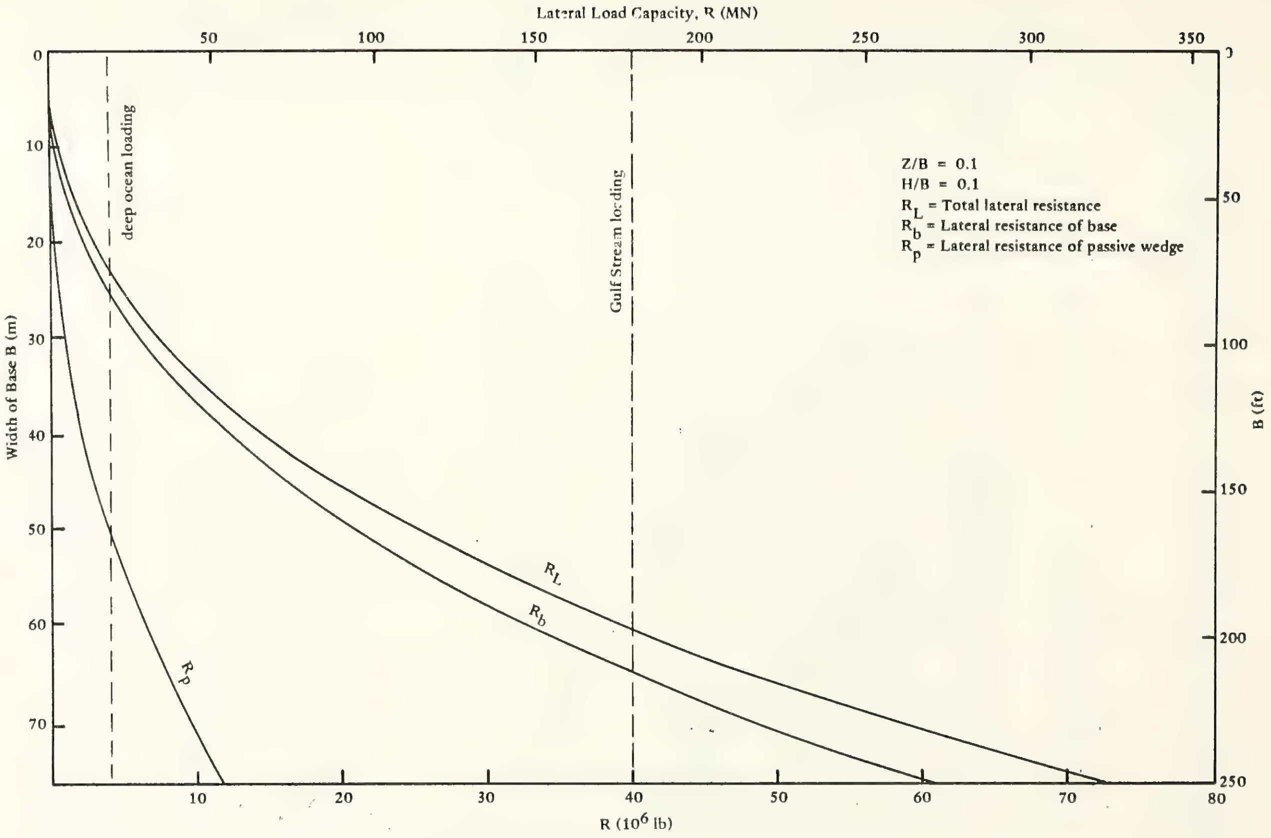


Figure 35. Lateral capacity of deadweight anchors versus width in soil category C.

2. Non-Cohesive Soil. The analysis of the passive wedge component for a non-cohesive soil (category D, sand) utilized the relationship

$$R_p^1 = 1/2 \gamma Z^2 K_p \qquad (14)$$

where K_p = coefficient of passive lateral earth pressure on the cutting edge

Table 14 presents values for the coefficient of passive lateral earth pressure on the wall of the leading cutting edge, along with a list of assumptions inherent in the use of these K_p values. The sand of category D was assumed to have an angle of internal friction, ϕ of 0.52 rad (30 deg), a conservative value. From Table 14 the K_p value for a $\phi = 0.52$ rad soil is 4.78. Conservatism has been interjected in the selection of $\phi = 0.52$ rad because of the unknown cutting edge performance in highly calcareous category D soils. The values of the effective friction angle of a calcareous soil on a cutting edge wall, δ, may be considerably reduced due to the soft nature of the carbonate grains relative to steel. The buoyant unit weight of the sediment, γ, was assumed as 5.5 kN/m^3 (35 pcf). The conservative value was used for two reasons: (1) soil in the passive wedge will be less dense than soil under the anchor, and (2) soil in the passive wedge might be removed by scour. Lateral load capacities derived from the passive wedge are presented in Figure 36.

Analysis of Base Shear Component.

1. Cohesive Soil. The deadweight anchor is assumed to slide on a soil failure plane passing through the tips of the cutting edge as shown in Figure 33. The resistance to lateral load developed along this plane in a cohesive soil is in its simplest form:

$$R_b = s \times A \qquad (15)$$

where A = bearing area of the deadweight (m^2).

Lateral load resistances calculated using (15) may prove somewhat unconservative in some more-sensitive seafloor sediments because the failure zone will probably develop in a progressive fashion with the realizable, average shear strength on the base shearing plane being somewhat lower than the measurable undrained shear strength of the sediment, s. No allowance for the progressive failure effect has been made in this preliminary effort. Undrained shear strengths used in the anlysis were obtained from the strength profiles of Figure 1 at soil depths equal to the cutting edge length.

Lateral load capacities arising from base shear in category A and C soils have been plotted as a function of anchor width in Figures 34 and 35 respectively. Note that in these figures the ratio of cutting edge length to anchor width, Z/B, has been fixed at 0.1.

Table 14. Coefficients of Passive* Lateral Earth Pressure on Deadweight Cutting Edge (after Tschebotarioff, 1962)

$\overline{\phi}$ (deg)	10	12.5	15	17.5	20	25	30	35	40
$\overline{\phi}$ (rad)	0.17	0.22	0.26	0.30	0.35	0.44	0.52	0.61	0.70
Kp	1.56	1.76	1.98	2.25	2.59	3.46	4.78	6.88	10.38

*Assumptions:
a. cutting edge wall is vertical
b. soil surface is horizontal
c. soil is non-cohesive, $s = 0$
d. angle of wall friction, δ, $= \frac{1}{2}\overline{\phi}$

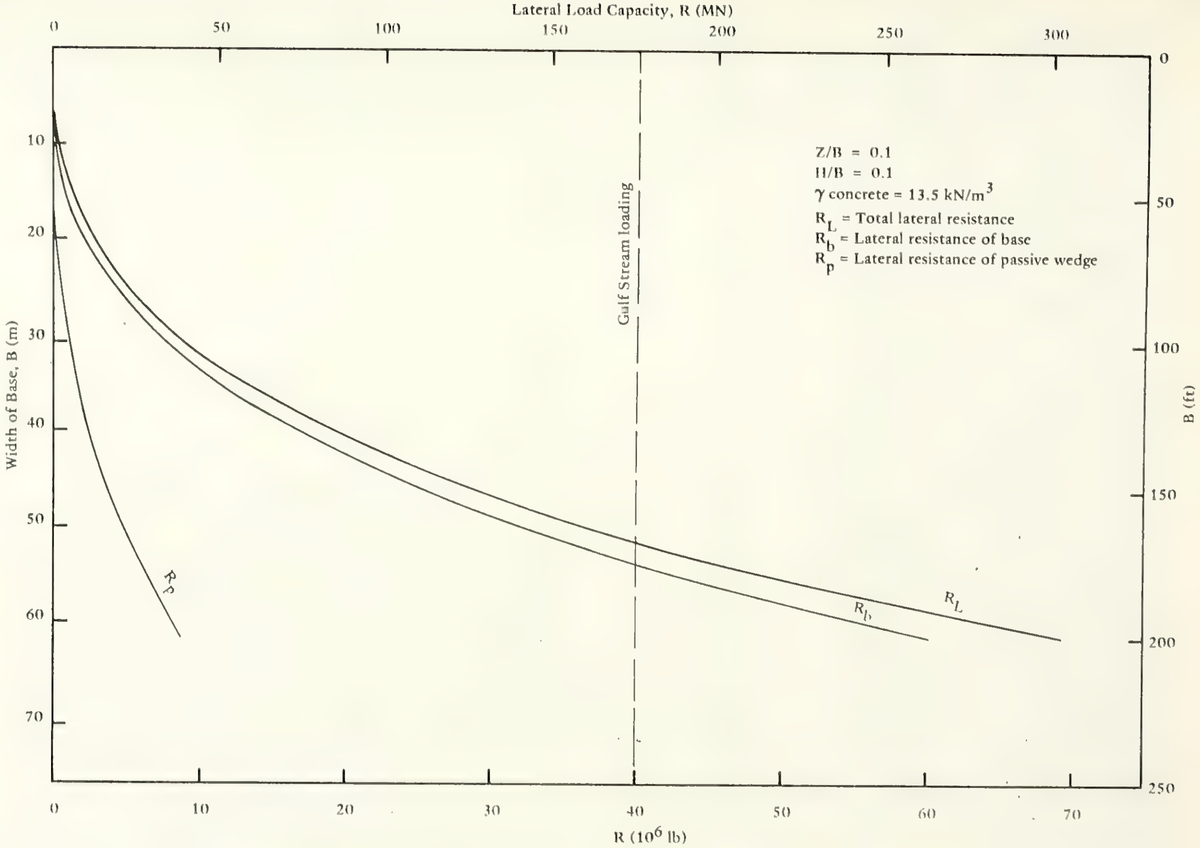


Figure 36. Lateral capacity of deadweight anchors versus width in soil category D.

Figures 34 and 35 also present the total lateral load capacity, R_L, the sum of the base shear and passive wedge components, for anchors of square plan. For the Z/B ratio of 0.1 treated and for the category A soil the passive wedge resistance, R_p, comprises about 20 percent of the total lateral load capacity, R_L. In the category C soil the passive wedge resistance comprises about 15 percent of the total lateral load capacity. The slight difference in the contribution of the passive wedge is due totally to differences in the shapes of the assumed soil shear strength profiles.

Increasing the Z/B ratio will increase the proportion of ultimate lateral load capacity carried by the passive wedge, R_p, because the passive wedge capacity increases as a function of depth squared, Z^2, while the base shear capacity increases nearly linearly with depth.

2. Non-Cohesive Soil. The resistance to lateral load developed along the base shear plane in a non-cohesive soil is:

$$R_b = W_{eff} \times \tan \bar{\phi} \qquad (16)$$

where W_{eff} = the effective weight or force on the base shear plane (N)

$$\text{or} \quad W_{eff} = (W + W_s) - P_v \qquad (17)$$

where W = submerged weight of the anchor (N)

W_s = submerged weight of the soil entrained within the cutting edges and above the base shear plane (N)

P_v = vertical force component applied to anchor by mooring line (N).

The lateral load capacity of a deadweight anchor on non-cohesive soil is then directly proportional to the effective weight on the base shear plane. The lateral load capacity can be modified by either (1) increasing the height of the deadweight to increase its mass or by (2) increasing the density of materials comprising the deadweight. The first option for increasing the effective weight per unit area, that of increasing the height of deadweight, is limited by overturning potential and by local bearing capacity potential. The second option, that of increasing the mass per unit volume of the deadweight, is limited by material availability.

Base shear capacity of the deadweight anchors for OTEC on a category D soil were calculated assuming an effective angle of internal friction of 0.52 rad (30 deg) and a soil submerged unit weight of 7.1 kN/m^3 (45 pcf). Figure 36 presents the lateral load component due to base shear, R_b, as a function of the anchor block width for a square anchor. The data presented is for an anchor block with cutting edges with a Z/B ratio of 0.1.

Figure 36 also presents the total lateral load capacity, R_L, of the deadweight anchor with Z/B = 0.1 on category D soil. For these conditions, the passive wedge resistance, R_p, comprises about 12 percent of the total lateral load capacity, R_L. The fact that the passive wedge is a minor contributor to the lateral load capacity is quite fortunate. The passive wedge, in some environments, especially in category D soils, could be substantially removed by scour. Because the wedge is a minor contributor, its contribution to the lateral load capacity can be neglected without drastically affecting the anchor block dimensions.

Results. The lateral load capacity analysis suggests sizes required for deadweight anchors in various soil categories. These are presented in Table 15. The results of this phase are significant when considering potential techniques for contsructing, transporting and installing deadweight anchors for OTEC. For example, if a deadweight anchor with a lateral load capacity of 18 MN (4x10^6 lbs) were required at a particular OTEC site with category C sediments (Figure 35), that lateral load capacity could be provided by a square-plan deadweight anchor 22 m (72 ft) on a side. This anchor could be transported in the well of the heavy-lift ship Glomar Explorer (well width 22.5 m (74 ft). However, deadweights of greater width could not be transported within the ship's well, and other transport techniques would be necessary.

Lateral Load Capacity as a Function of Cutting Edge Length and Anchor Width

Assumptions. One possible way of increasing the lateral load capacity of an anchor block of given plan dimensions is to increase the length of the cutting edge. Increasing the length and embedment of the cutting edges moves the base shear plan downward into presumably stronger material and increases the size and load capacity of the passive wedge. Unfortunately, the lever arm of the overturning moment is also increased, and thus the lateral sliding failure mode may no longer be controlling.

The review of lateral load capacity as a function of relative cutting edge length, i.e., Z/B ratio, assumes that lateral sliding would be maintained as the dominant failure mode. This failure mode can be maintained at the expense of restricting the direction of load application to only one direction, i.e., by restricting the multi-direction anchor to a uni-direction anchor. Reduction in the overturning moment is achieved by lowering the mooring line attachment point as illustrated in Figure 37, or by re-distributing the anchor mass to better counter the overturning moment. The uni-direction anchor is suitable for use in multi-point moorings and in areas of uni-directional loading (e.g., moorings near the axis of the Gulf Stream).

Results. Lateral load capacity predictions for deadweights with Z/B ratios other than 0.1 were calculated using the same assumptions and techniques employed above for deadweights with Z/B = 0.1. The results are presented in Figures 38, 39, and 40 for Soil Categories A, C, and D respectively.

Table 15. Length of Side of Square-Plan Deadweights Required to Satisfy Lateral Load Requirements, for $Z/B = 0.1$, Line Angle $\beta = 0$, $H/B = 0.1$

Soil Category	Lateral Load Required, R_L		Length of Side, B	
	(MN)	(10^6 lbs)	(m)	(ft)
A	18	4	40	130
B	18	4	27	90
C	18	4	22	72
	180	40	60	197
D	180	40	57	186

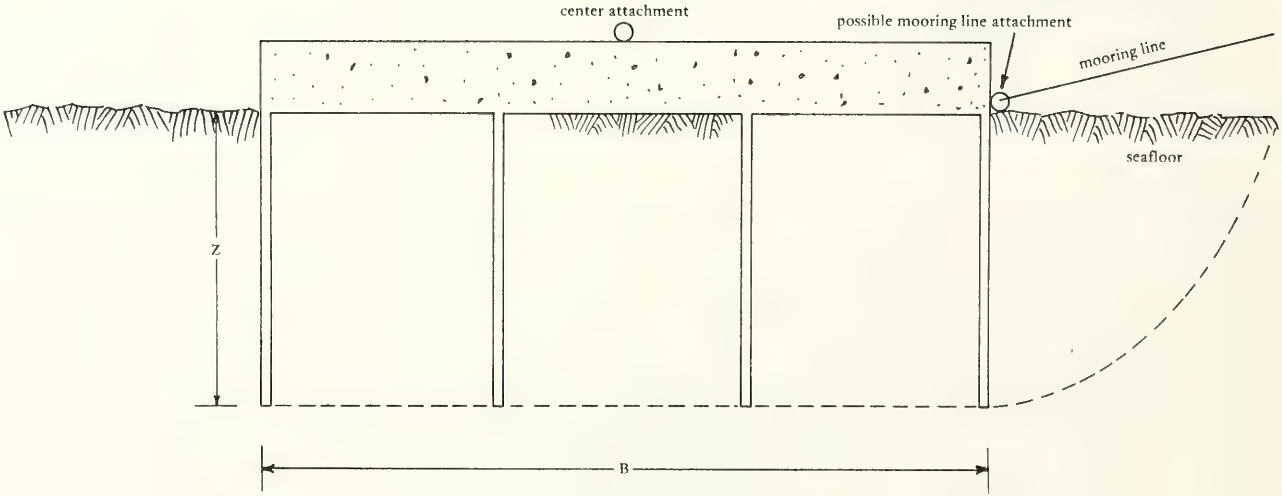


Figure 37. Off-center mooring line attachment point, used to decrease overturning moment.

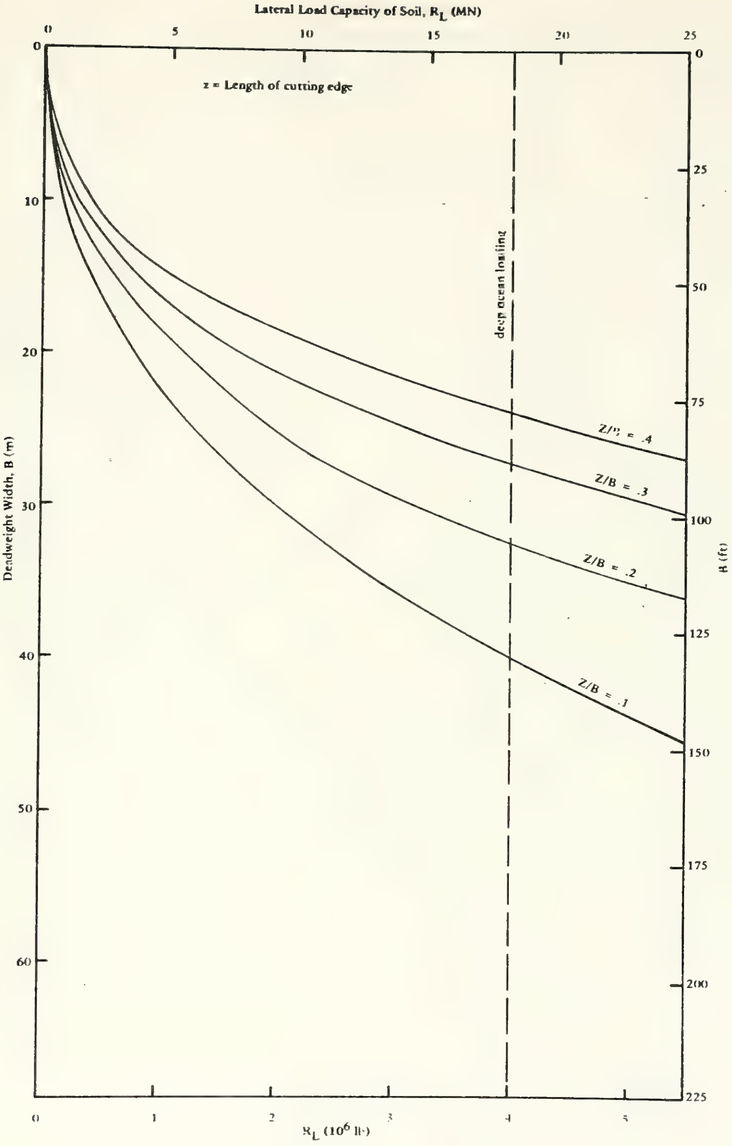


Figure 38. Lateral load capacity of deadweight anchor for various Z/B ratios in soil category A.

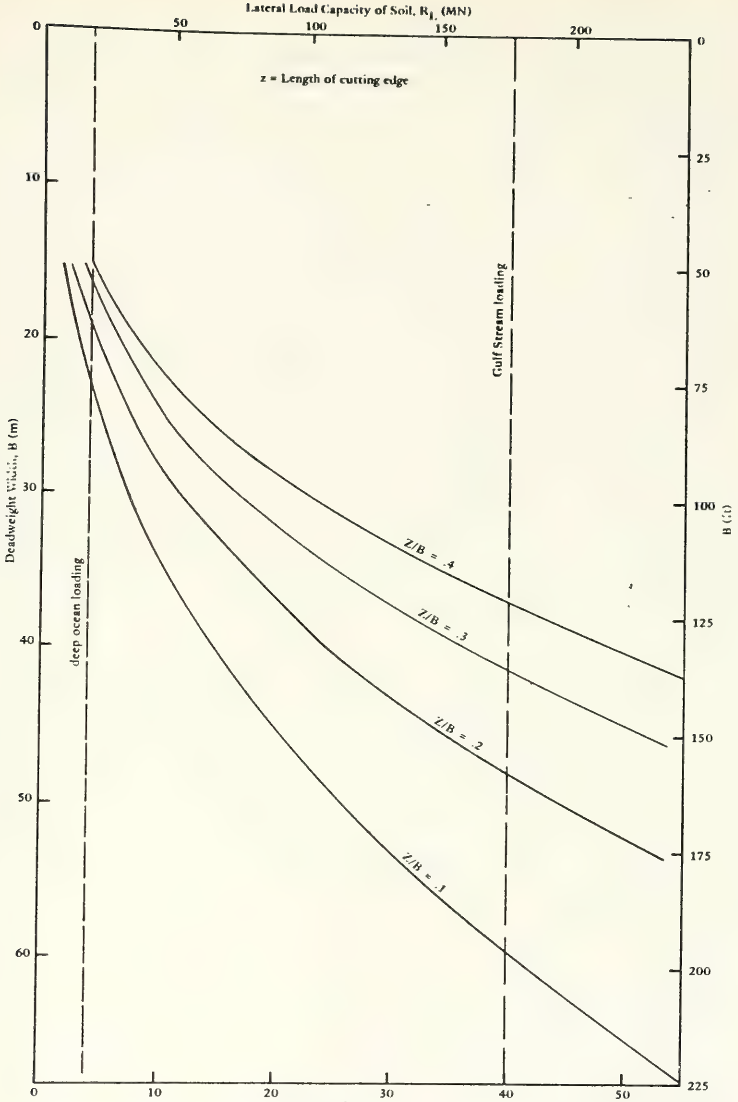


Figure 39. Lateral load capacity of deadweight anchor for various Z/B ratios in soil category C.

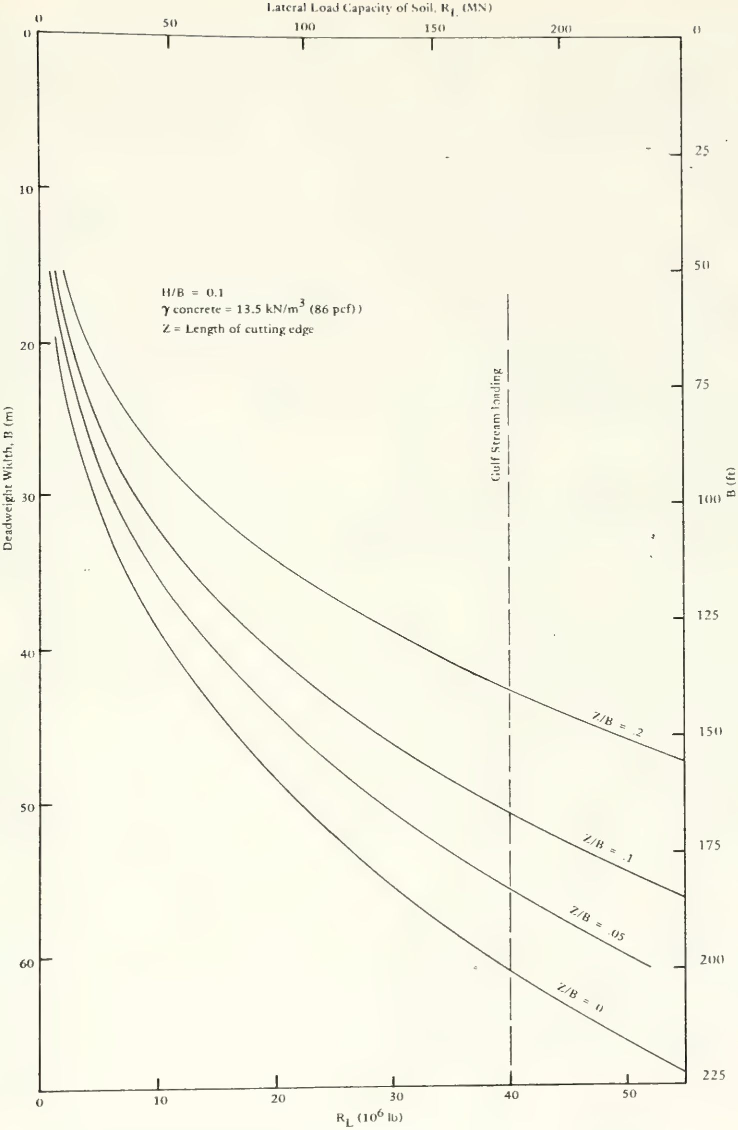


Figure 40. Lateral load capacity of deadweight anchor for various Z/B ratios in soil category D.

Optimal Ratios of Cutting Edge Length to Anchor Width

Purpose. The previous section addressed increasing the lateral load capacity of a deadweight by increasing the cutting edge length. Cutting edge lengths are limited in practice; however, by two factors. One of these factors, aggravation of the overturning problem was treated earlier. The other, implementation of cutting edge penetration, places efficiency/cost limitations on the cutting edge lengths and section thicknesses. Lengthening the cutting edges increases the lateral load and the effective moment arm acting to bend the cutting edge at its point of attachment to the anchor base (see Figure 41). To develop the increased bending moment and shear capacity in the cutting edge, the structural section must be increased (thickened). The cutting edge of greater thickness and length requires a greater force, obtained from the deadweight submerged weight, to accomplish complete embedment. The increased costs of the greater structural section in the longer cutting edge and the increased costs for materials, construction, and handling of the heavier deadweight both impose an upper limit on the practical length of cutting edge to be used.

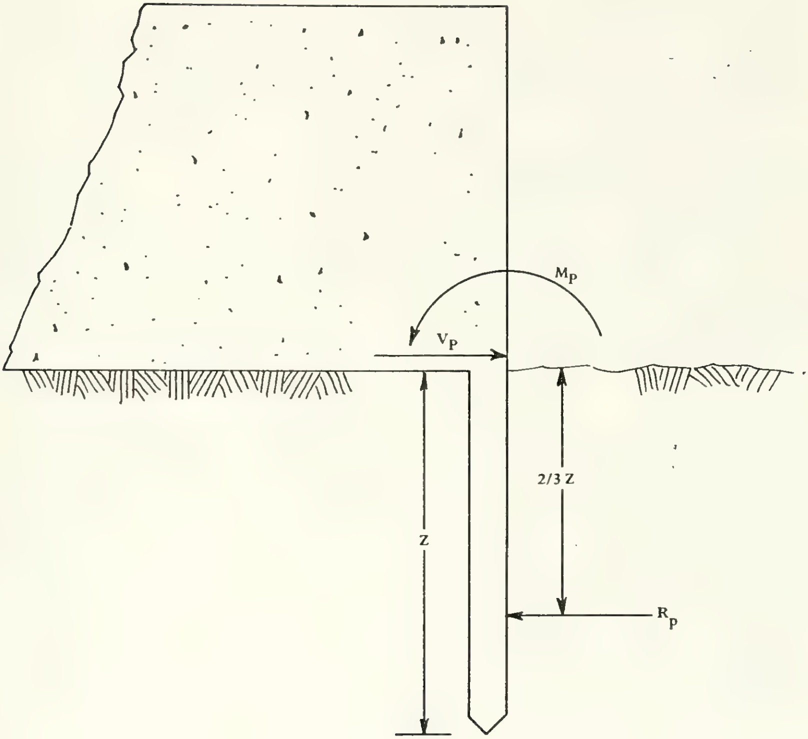
This report section approaches the subject of optimal Z/B ratios from an idealized standpoint. It assumes that minimum cost is directly related to the minimum deadweight mass required to embed a cutting edge design sufficient to resist the design lateral load. Clarification of this statement is left to the discussion of results for this section.

Structural Design of Cutting Edges.

1. Procedure. The cutting edges were designed to resist the pressure of a passive wedge as the cutting edge is moved laterally without rotation. The pressure distribution on the cutting edge wall was assumed triangular. Then the force resultant acting on the wall could be taken to operate at the two-thirds point (see Figure 41). The force per unit width of anchor block R_p^1 was obtained from the earlier presented passive wedge data. The force times the moment arm yields the bending moment to be resisted at the juncture of cutting edge and deadweight block:

$$M_p^1 = R_p^1 \ (2/3\ Z) \tag{18}$$

This analysis is conservative, but justified because it is simple. The analysis is conservative because the cutting edges would normally be arranged in a grid pattern beneath the deadweight and would interact. Those cutting edges oriented parallel to the mooring line direction would act as shear walls assisting in transferring the mooring load to the perpendicular oriented set of cutting edges and thereby reducing the bending moment in that perpendicular set.



$$M_p = 2/3\ ZR_p$$

Figure 41. Force-moment diagram for skirt.

2. Results. Four potential configurations and loading conditions were selected, the bending moments per unit width, M_p', calculated, and then reinforced concrete and strucutral steel wall sections were designed to resist the applied bending moment. Input parameters and calculated wall thicknesses are contained in Table 16. The design assumed that full loading could be applied to either side of the cutting edge. The reinforced concrete design utilized a working stress design and incorporated 76 mm (3 in.) of cover over the steel. The steel cutting edge design also used a working stress design with efforts to minimize the wall thickness, i.e., the section is not the most economical for resisting the applied moment. The steel section was assumed composed of a row of H-pile sections joined into a wall and stabilized by two steel sheets, one on the inside flange, the other on the outside. The design thicknesses, t, for the cases analyzed were plotted and joined in Figure 42 to provide a thickness estimating curve for cutting edges.

Please note these thicknesses, t, are based on a working stress design while the passive resistances noted are ultimate loadings. Therefore, a change in technique for cutting edge thickness design may be appropriate.

Load Required to Penetrate Cutting Edges.

1. Assumptions. The critical factor in dealing with cutting edge penetration is the soil shear strength assumed to be acting. In the treatment of cutting edge penetration, the tip of the cutting edge was assumed to be penetrating in a bearing capacity failure mode with the soil shear strength being mobilized being equal to the undistrubed shear strength (see Figure 1). The walls of the cutting edge were assumed to be sliding downward in partially remolded soil. The degree of remolding was assumed to be at least as great as the minimum value of sensitivity quoted for that soil category (see Table 1 for sensitivities). The penetration of the cutting edge was assumed to reduce the shear strength, s, of a category A soil by a factor of 2, a category B soil by a factor of 3, and a category C soil by a factor of 2. The curves of soil side friction, s_f, obtained by this technique are compared in Figures 43, 44, and 45 with the recommended values, $\gamma (\bar{\sigma}_m + 2s_m)$, to be used for estimating the soil side friction on driven steel piles (McCelland, 1974).

2. Analysis. The cutting edge tips were assumed to be blunt. Penetration of this blunt tip at the expected low emplacement velocities is described by a conventional bearing capacity analysis. This approach is conservative because every attempt will be made to streamline (sharpen) the tips to reduce the tip resistance. However, unknown dynamic effects may negate any strength reductions due to streamlining, therefore, no attempt is made in the analysis to streamline the cutting edge tips.

The walls of the cutting edges were assumed not specifically prepared, that is, smoothed or lubricated.

Table 16. Thickness of Cutting Edges for Selected Cases

Case	1	2	3	4
Soil Category	A	A	C	C
Anchor Width, B(m)	43	26	60	37
Cutting Edge Length, Z(m)	4	10	6	15
Z/B	0.1	0.4	0.1	0.4
Laterial Load on Edge per Unit Width R_p^1 (kips/ft) (kN/m)	5.18 75.60	35.0 510.8	31.3 456.8	131.3 1916.2
Maximum Bending Moment per Unit Width M_p^1 (kip ft/ft) (m.MN/m)	48.700 0.217	784.50 3.49	413.00 1.84	4217.0 19.0
Cutting Edge Thickness Required, t Concrete (m) Steel (m)	 0.41 0.20	 1.27 0.61	 0.92 0.46	 2.69 1.40

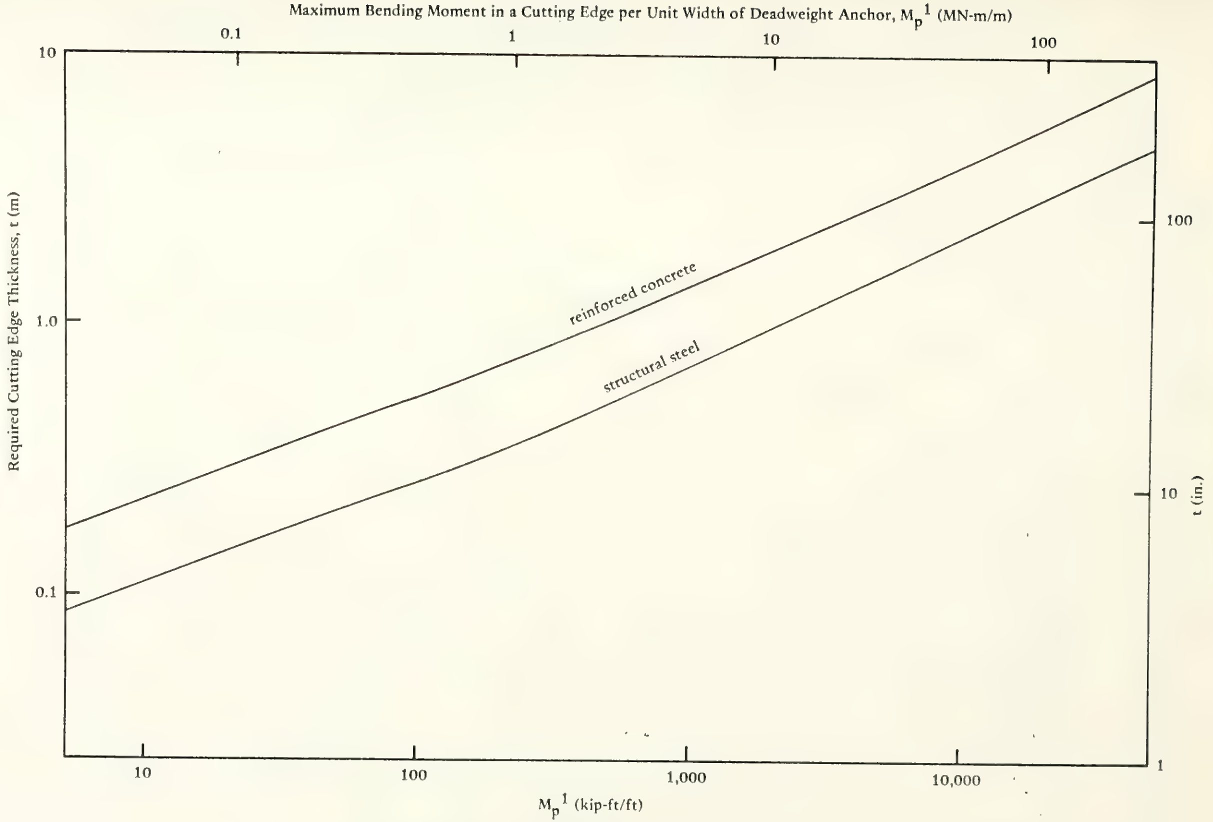


Figure 42. Estimate of cutting edge thickness required to resist the bending moment at the cutting edge juncture with the anchor block.

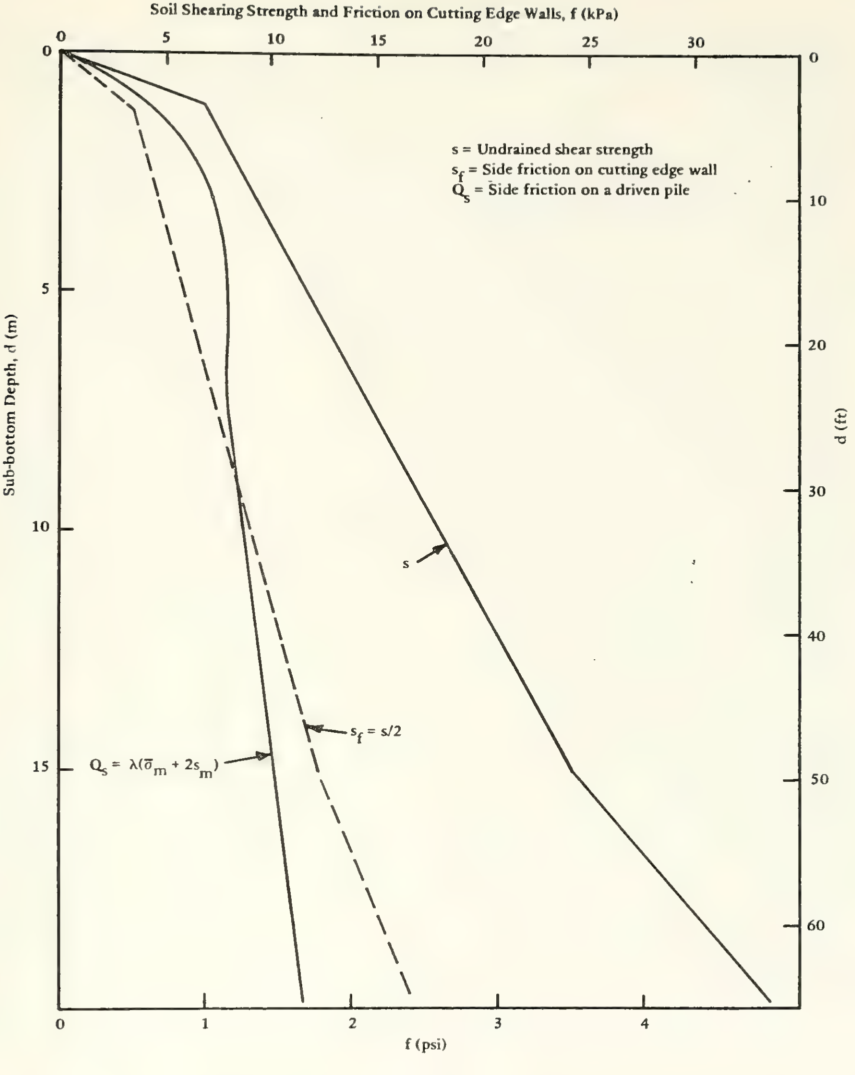


Figure 43. Comparison of soil friction values for cutting edge penetration analysis in soil category A.

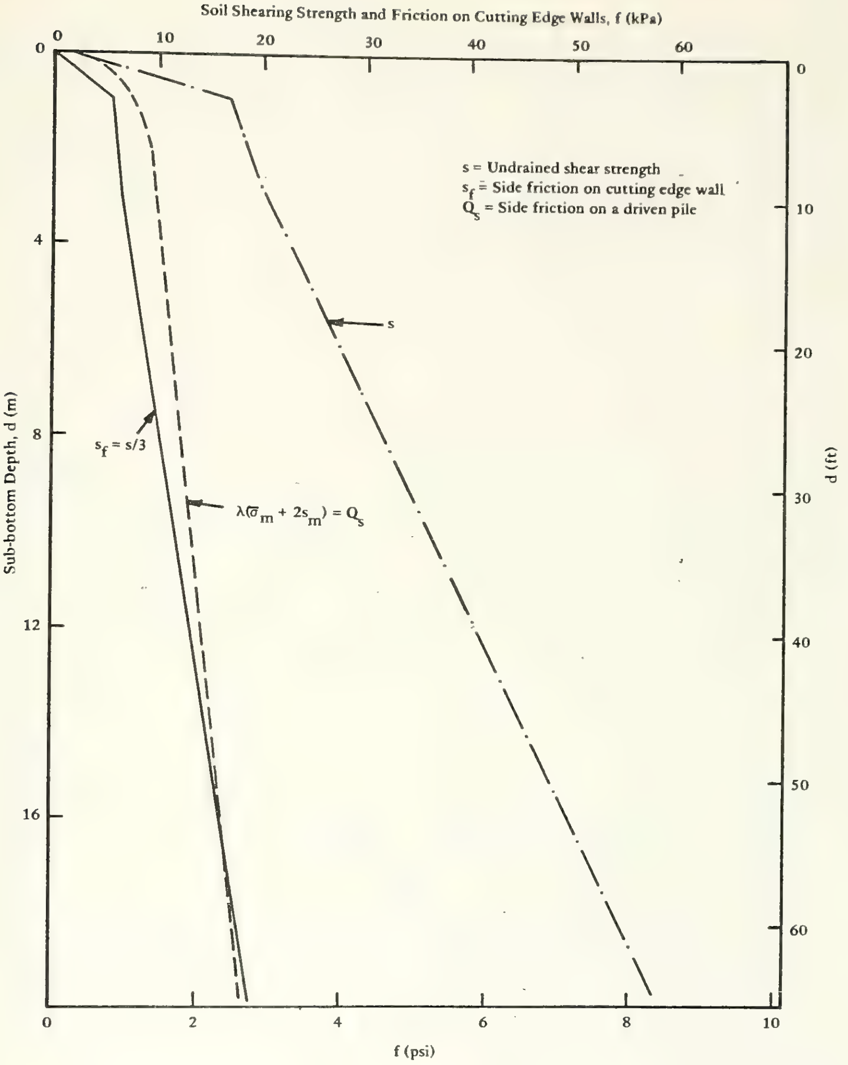


Figure 44. Comparison of soil friction values for cutting edge penetration analysis in soil category B.

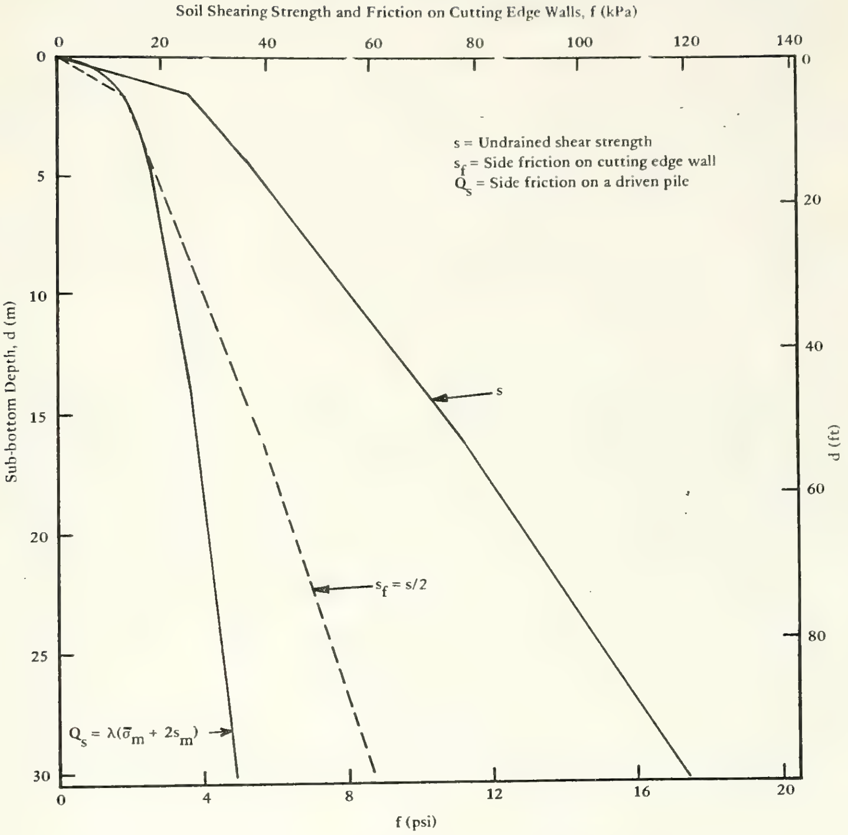


Figure 45. Comparison of soil friction values for cutting edge penetration analysis in soil category C.

a. Cohesive Soils. The force required to embed the deadweight skirts, Q_e, is to be evaluated using the relationship

$$Q_e = sN_cA + s_fA_s \qquad (19)$$

where s = undrained shear strength at the cutting edge tip, (Pa)

N_c = bearing capacity factor taken to be 9 for cutting edges of $Z/t = 5$

A = bearing area of the cutting edge tips (m^2)

s_f = mobilized soil/cutting edge wall friction (Pa)

A_s = side area of the cutting edge (m^2)

Unfortunately, during the cutting edge embedment analysis, the bearing capacity term of Equation (19), i.e., the term sN_cA, was improperly evaluated yielding values of force required to embed the cutting edges, Q_e, about 5 percent too high. The error is on the conservative side indicating a greater deadweight required than is really necessary to accomplish embedment.

b. Non-Cohesive Soils. The force required to embed the deadweight skirts in non-cohesive soils (sand-like) is to be evaluated using the relationship

$$Q_e = A(1/2\ \gamma t N\gamma d\gamma + ZN_qs_qd_q) + A_sK\bar{\sigma}_v\tan\phi \qquad (20)$$

where t = thickness of cutting edge (m)

$N\gamma$ and N_q = bearing capacity factors (see Figure 40 of CEL TM-42-76-1, Valent et al., 1976)

s_q = shape factor = $1 + \tan\phi$

d_q = factor to account for the shearing resistance of soil above the bearing plane of the anchor

$$= 1 + 2\tan\phi(1 - \sin\phi)^2\tan^{-1}(Z/t) \qquad (21)$$

$$d_\gamma = (\text{purpose same as } d_q) = 1 + 0.4\tan^{-1}(Z/t) \qquad (22)$$

K = coefficient of lateral pressure on cutting edge wall

$\bar{\sigma}_v$ = vertical effective stress (Pa)

After having used the above relationships, (20), and (21) in development of the forces required for penetration, questions have arisen regarding the appropriateness of the magnitudes used for many of the terms. As will be shown later, it is not appropriate to spend a great deal of OTEC effort in resolving uncertainties regarding embedment of cutting edges in sand, at least at this stage of the effort. Later presentations will show that the most efficient deadweight design on sand will use only very shallow cutting edges.

3. Results, Force Required to Embed. Steel cutting edges were given the greater attention because they are only one-half the thickness of concrete cutting edges and thus require significantly less force to embed. The forces required for embedment, Q_e, were calculated for various sizes of square blocks (widths = B) and for Z/B ratios of 0.1, 0.2, 0.3, and 0.4 on cohesive soils and 0.05, 0.10, 0.15, and 0.20 on non-cohesive soils. Intermediate cutting edges were placed beneath the anchor blocks to ensure development of a planar failure plane beneath the anchor. The ratio of cutting edge spacing to length, b/Z, varied with the soil type: ratios of b/Z = 2, 3, 5, and 6 were used in Soil Categories A, B, C, and D respectively. The bearing areas and wall areas of all cutting edges were summed to yield the A and A_s areas respectively.

Figures 46, 47, 48, and 49 present the forces required to embed cutting edges in Soil Categories A, B, C, and D respectively. The forces for embedment are represented as solid lines. The dashed curve on these plots will be explained in the subsequent seciton.

Optimum Cutting Edge Ratios.

1. Approach. After the development of data above showing the submerged weights necessary to embed cutting edges for deadweight anchors for varying B, Z/B, b/Z, and soil strength profiles, it was then of interest to look for optimum cutting edge characteristics. First, optimum cutting edge characteristics were defined as those that would provide the required lateral load capacity for the minimum required embedment load, Q_e. Given this definition, it was then necessary to develop a curve of loci of cutting edge characteristics capable of resisting the required lateral load, i.e., 18 MN or 180 Mn (4×10^6 lbs or 40×10^6 lbs) depending on the environment. The curve of loci is a function of Z/B and B and is represented as a dashed line on Figures 46 - 49.

The method for developing this curve of loci was to first enter Figures 38, 39, and 40 (and one other unpublished curve for category B soil) and pick out B and Z/B parameters capable of resisting the required loading. With these values for the respective soil category, Figures 46 through 49 were entered from the B axis until the appropriate Z/B curve was encountered. Thus the loci of cutting edge characteristics capable of resisting the lateral loading were defined.

2. Results. Figure 46 indicates for a category A soil there is not much difference in performance for different anchor-block/cutting-edge associations. Please note this evaluation does not touch on the subject of overturning potential which becomes worse with increased Z/B ratio.

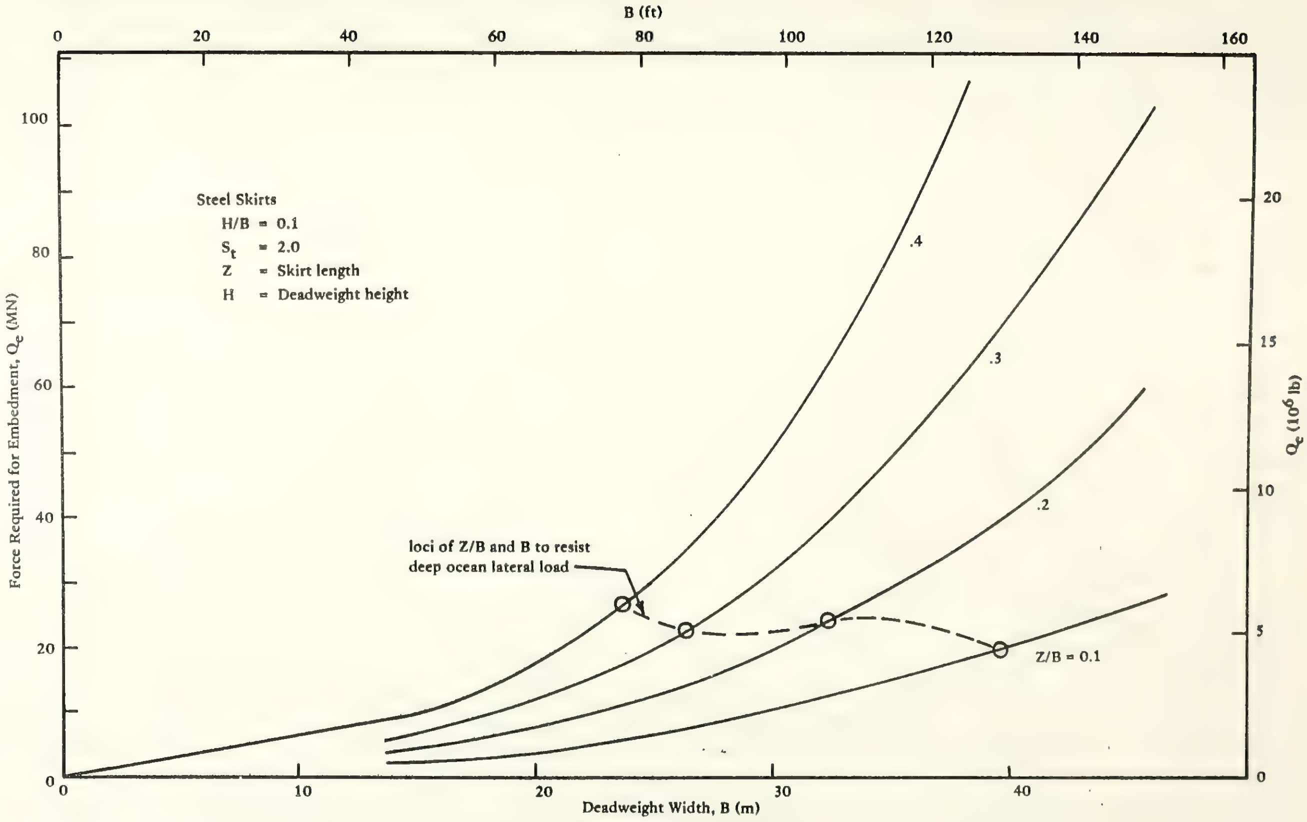


Figure 46. Required embedment force as a function of deadweight width and height in soil category A.

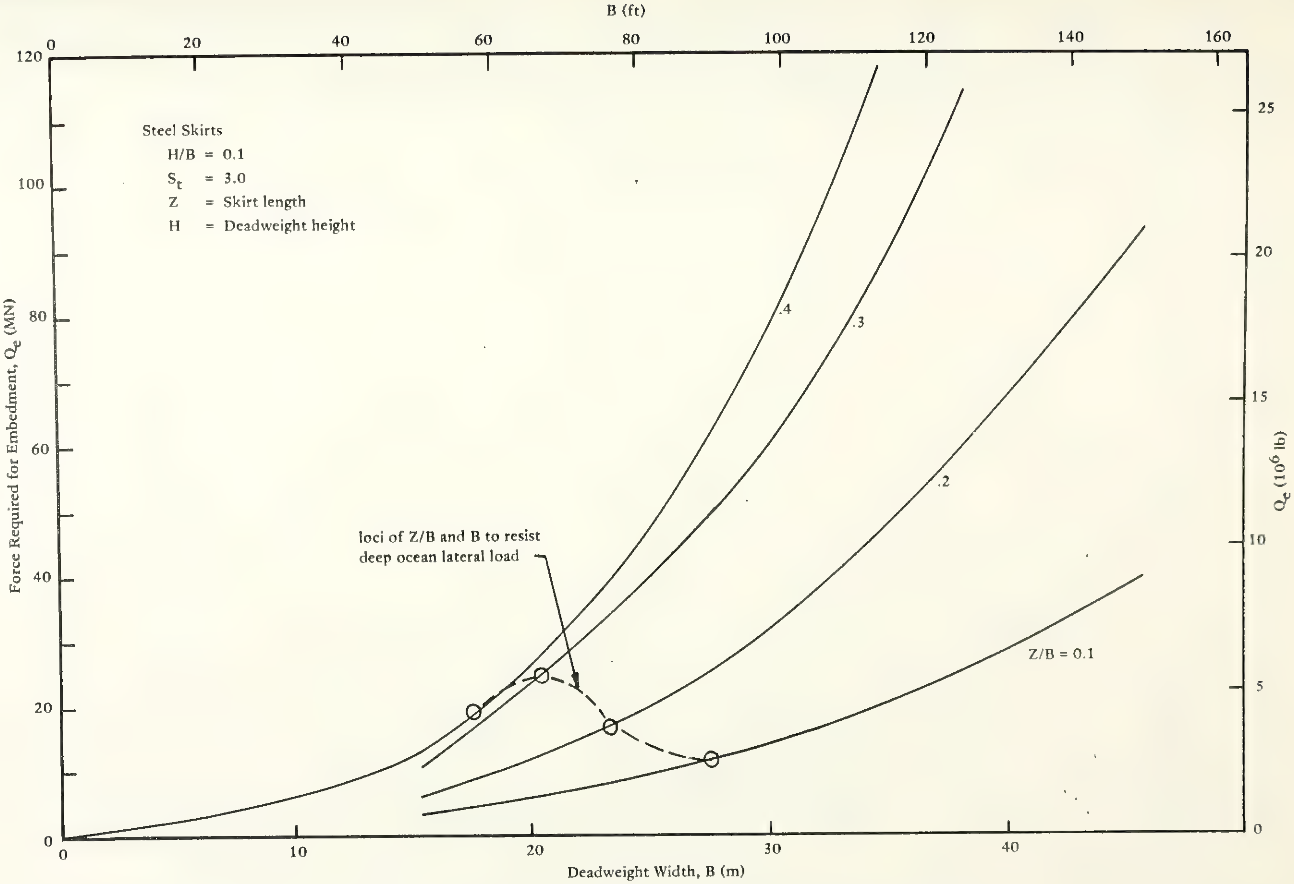


Figure 47. Required embedment force as a function of deadweight width and height in soil category B.

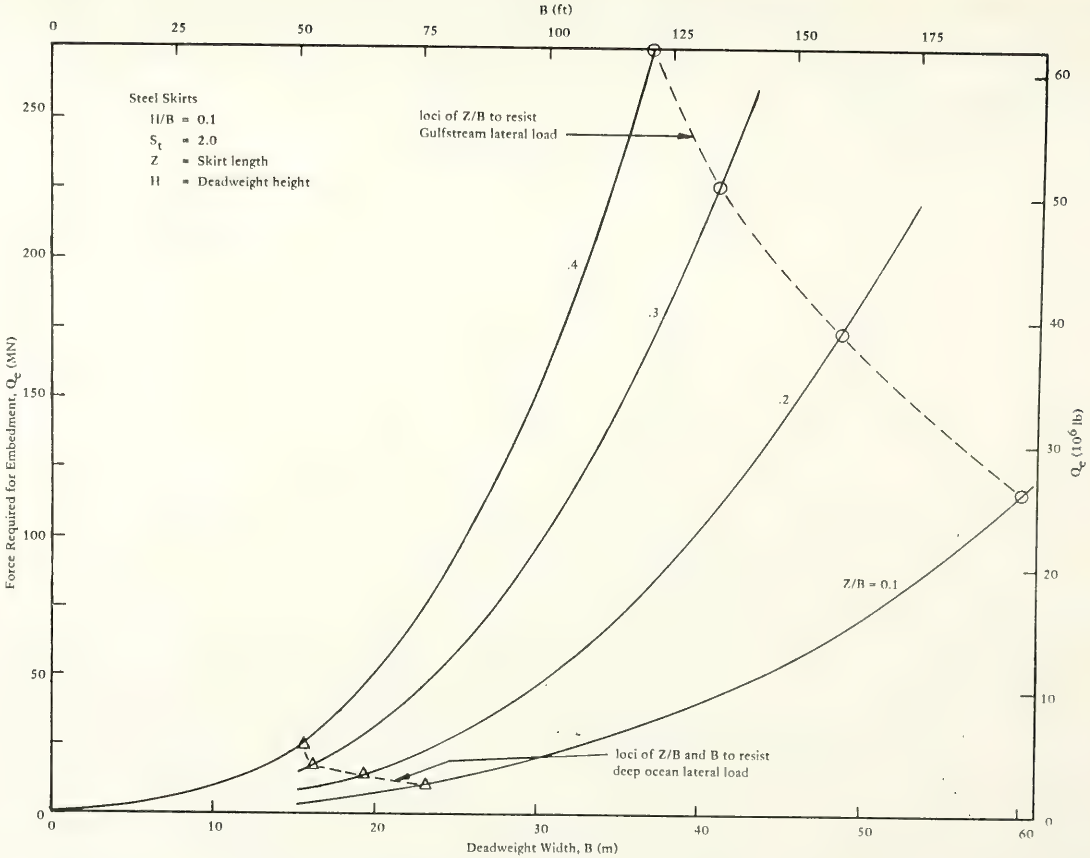


Figure 48. Required embedment force as a function of deadweight width and height in soil category C.

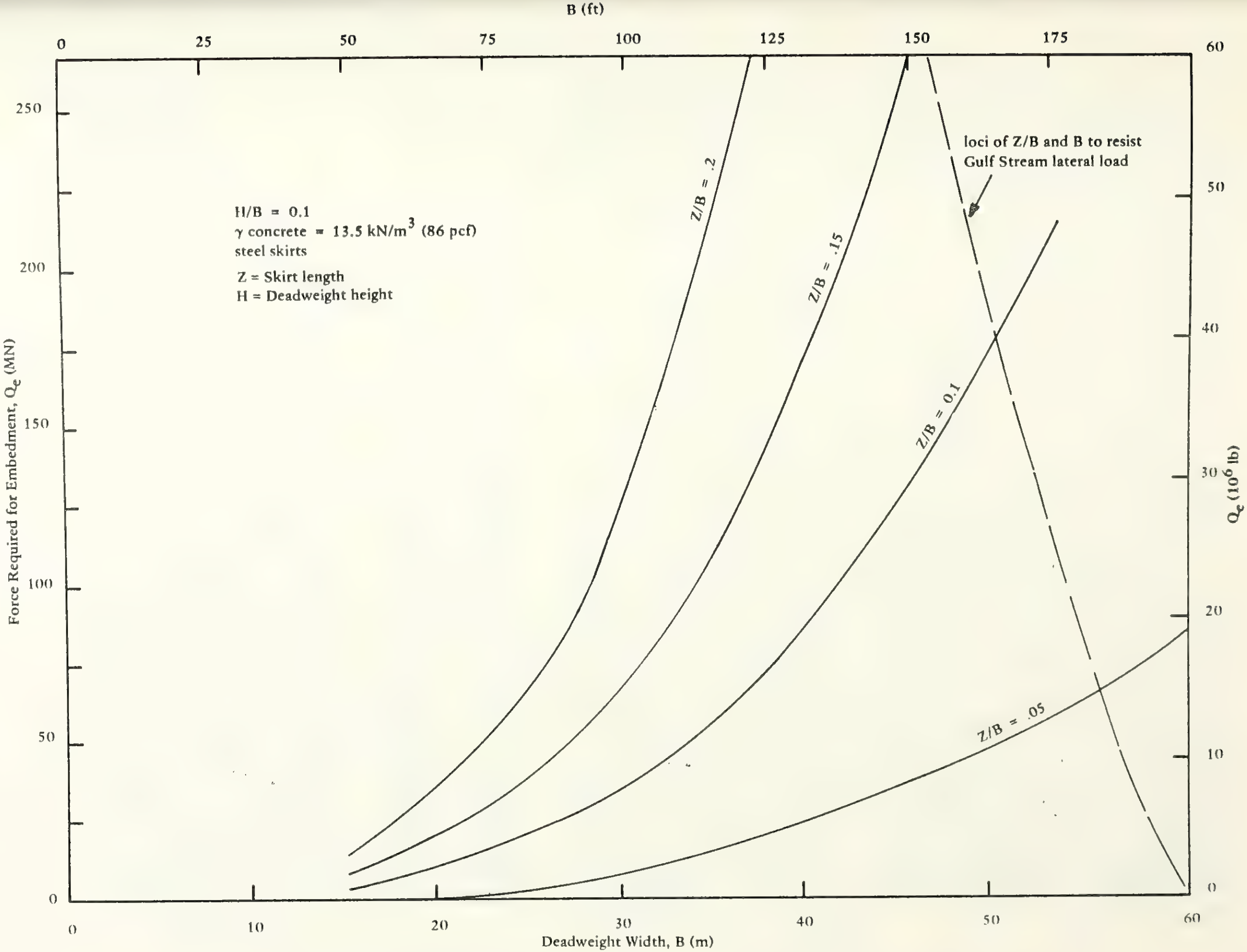


Figure 49. Required embedment force as a function of deadweight width and height in soil category D.

Figure 46 indicates that a deadweight weighing between 20 and 25 MN (4.5×10^6 to 5.5×10^6 lbs) will be required to drive a cutting edge system necessary to resist the deep ocean 18 MN lateral load. The waviness of the dashed curve is due to change in the number of intermediate cutting edges in order to maintain a $b/Z < 2$. As indicated before, the cutting edges must be maintained this close together to cause the lateral sliding failure to develop in the horizontal plane of the cutting edge tips.

Figure 47 indicates that the minimum embedment force is required for a cutting edge length to anchor width ratio of 0.1. Embedment force is actually the weight required on the seafloor (for zero line angles). Thus, by minimizing embedment force, weight is minimized and efficiency is maximized. This figure demonstrates that efficiency is maximized at the smaller cutting edge lengths. This trend is very favorable in that overturning should not be a problem with such a wide and squat anchor unit. Figure 47 indicates a required embedment force of 11 MN in order to realize the 18 MN lateral load capacity.

For a category C soil, Figure 48 indicates again that shorter cutting edges are more efficient, in fact, the optimum Z/B ratio is probably less than 0.1. For the deep ocean environment, a cutting edge embedment force of 11 MN is required to achieve the 18 MN horizontal holding capacity. The subject anchor block would be about 22 m (72 ft) on a side. For the Gulf Stream environment, a cutting edge embedment force of 117 MN (26×10^6 lbs) will be required to drive the required skirts to achieve the required 180 MN lateral load capacity. This driving force cannot be placed on the seafloor at one time because the largest lift system known can handle only 62 MN, half of that necessary, and that load only to 1500 m water depth. This limitation in load handling capability suggests that a deadweight anchor in category C soil subjected to a Gulf Stream loading would necessarily:

a. Incorporate a sophisticated ballasting system to increase the submerged weight of the anchor once it was on the seafloor in order to fully embed the cutting edges.

b. Incorporate a means for adding mass on top of the anchor to help weight the anchor and drive the cutting edges in a uniform manner.

c. Incorporate a second anchor system to provide the additional lateral load capacity required, a system such as lateral load resisting anchor piles emplaced through the anchor block.

Figure 49 indicates that for a category D soil (sand) the most efficient section in terms of required submerged weight required for embedding skirts is that with no cutting edges. This phenomenon occurs because the lateral load capacity of a deadweight on a non-cohesive (sand) material is predominantly influenced by the in-water weight of the anchor block. The use of cutting edges to drive the failure zone deeper in the sediment does little to increase the lateral load capacity.

However, cutting edges are desirable, at least around the periphery of the deadweight, to prevent undercutting of the anchor block by scour. Further, the substantial submerged weight must be made available anyway so that weight should also be used to drive some minimal cutting edge design. The base shear design assumes a square plan anchor block of side length B and height H equal 0.1 x B with the block material having an average submerged unit weight, γ, of 13.5 kN/m^3 (86 pcf).

Figure 49 indicates that a deadweight block with a side length, B, of 56 m (184 ft), a Z/B ratio of 0.05, and other parameters as above would resist the 180 MN lateral loading of the Gulf Stream. The force required to embed the cutting edges is 64 MN (14.5x10^6 lbs), for all practical purposes the same as the 62MN load capability of the Glomar Explorer. In practice the 64 MN, 56 m square deadweight with 2.8 m long cutting edges would consist of a structural prestressed concrete compartmentalized box placed on the bottom by a heavy lift system. The cutting edges of this box would be embedded uniformly by the submerged weight of the empty box. Given the box weight of 64 MN and the prestressed concrete unit weight in seawater of 13.5 kN/m^3 (86 pcf), the volume occupied by the prestressed concrete would be 4,800 m^3 (169,000 ft^3). The 56 m x 56 m x 5.6 m high box would enclose a volume of 15,300 m^3 (623,000 ft^3) of which 70 percent would be void space at the time of embedment. This void space would then be filled with a material selected to provide the necessary submerged weight (1) to maintain the deep, planar zone of sliding, (2) to resist the overturning moment, and (3) to resist some small uplift load from a non-zero mooring line angle. If the box were entirely filled with concrete, its submerged weight would be 240 MN (54x10^6 lbs), sufficient to resist overturning and some small amount of uplift component.

Summary. The lateral load and skirt embedment analyses together then have resolved the general shaping and sizing of the deadweight anchor system for OTEC. Anchors on cohesive sediments will require cutting edge length, Z, of 0.1B; those on non-cohesive sediments require shorter cutting edges to facilitate embedment with Z = 0.05B appropriate. Emplacement of anchor sizes sufficient to resist the deep ocean lateral loading of 18 MN (with near zero mooring line angle) is feasible with available technology and equipment. Emplacement of anchors sufficient to resist the Gulf Stream lateral loading of 180 MN (with near zero mooring line angle) on a category D soil (sand-like) is also possible today. Only the anchor conceptual design for resisting the Gulf Stream loading on a category C soil (clay-like) remains a significant problem because of the difficulty of properly applying driving force to uniformly embed the cutting edges. Viable concepts for resisting heavy loads on clay have been suggested but remain to be evaluated and compared.

Bearing Capacity Analysis

Introduction. The remaining section of this chapter briefly presents bearing capacity data. In general, the results of this section suggest

that practical deadweight anchors for OTEC in most deep ocean environments will not fail in a bearing capacity mode. However, deadweight anchors on non-cohesive sediments (sands) are susceptible to bearing capacity failure. This section will delve into the reasons for this low bearing capacity on sand, will indicate the magnitude of the capacity reductions, and will delineate steps necessary to make the anchors adequate.

Analysis. The vertical load capacity of a very large, shallow, horizontal bearing surface is calculated using the following relationships:

$$Q = B'L'(0.5\gamma\, B'N_\gamma s_\gamma d_\gamma i_\gamma k_\gamma + \gamma Z(N_q - 1)\, s_q d_q i_q k_q + sN_c s_c d_c i_c k_c) \qquad (23)$$

where $B' = B - 2e_1$

e_1 = eccentricity of the load resultant parallel to the B dimension (B is the short dimension)

$L' = L - 2e_2$

e_2 = eccentricity of the load resultant parallel to the L dimension (L is the long dimension)

s_q, s_γ, s_c = factors to account for anchor plane shape

d_q, d_γ, d_c = factors to account for the shearing resistance of soil above the bearing plane of the anchor

i_q, i_γ, i_c = factors to account for inclination of the resistant load

k_q, k_γ, k_c = factors to account for compressibility of the supporting soil

These factors are all discussed and relationships for evaluating them are presented in CEL TM 42-76-1 (Valent et al., 1976). The values assigned to these factors for the bearing capacity analysis of square deadweight anchors are presented in Table 17.

Some uncertainty surrounds the selection of the inclination and the compressibility factors. Both sets of factors deserve further experimental verification. The inclination factors i_q and i_γ are developed from the expressions presented in TM 42-76-1 for a non-cohesive soil as a function of P_HR_V (Figure 50). The authors note that the expressions developed are simplified and may not be totally valid for values of P_H/R_V, approaching 1.0.

Table 17. Values Assumed for Bearing Capacity Factors in the OTEC Deadweight Analysis

Cohesive Soils, Categories A, B, and C, $\bar{\phi} = 0$

$N_c = 5.14$, $N_\gamma = 0$, $N_q = 1.00$

(Note: therefore N_γ term of bearing capacity equation = 0.)

$s_c = 1.2$ $s_q = 1.00$
$d_c = 1.05$ $d_q = 1.00$
$i_c = 0.50$ $i_q = 1.00$
$k_c = 1.00$ $k_q = 0.60$

Non-Cohesive Soils, Category D

$s = 0$ $\bar{\phi} = 0.52$ rad (30 deg),

$\gamma = 6.75$ kN/m^3(43 pcf)

$N_\gamma = 22$ $N_q = 18$
$s_\gamma = 0.6$ $s_q = 1.5$
$d_\gamma = 1.0$ $d_q = 1.03$

For $R_V = 2 \times P_H$

$i_\gamma = 0.07$ $i_q = 0.19$
$k_\gamma = 0.6$ $k_q = 0.6$

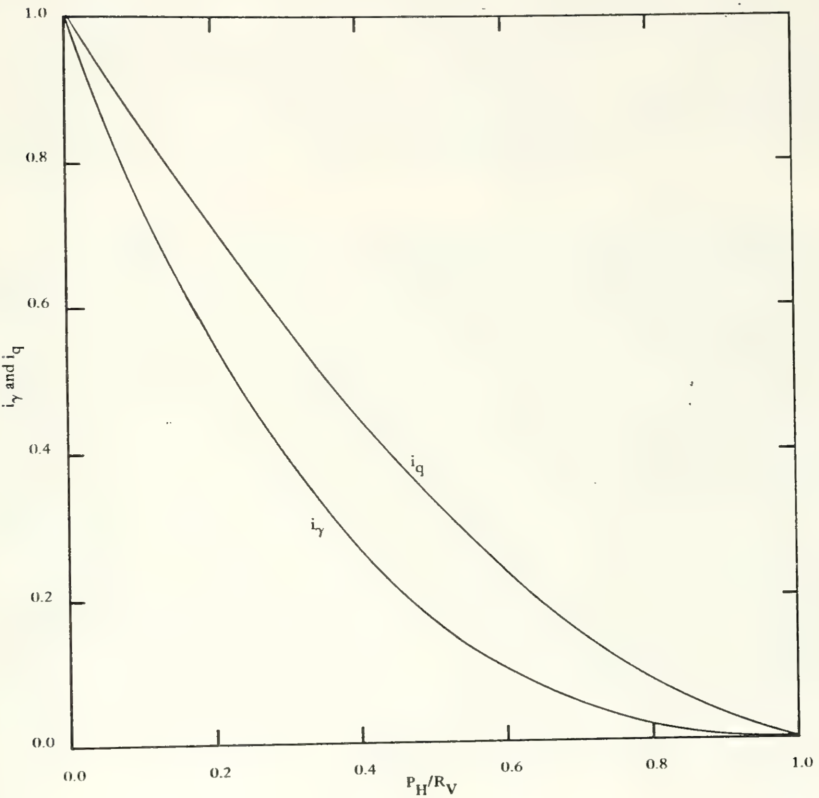


Figure 50. Bearing capacity coefficients for inclined loads soil category D.

The inclination factor i_c presents a problem in that at least three expressions are available yielding widely differing values for the loading condition of interest. Values vary from 0.25 to 0.7. The authors have relied on the expression of Hansen (1970) for the plane strain condition yielding $i_c = 0.5$ for the OTEC anchor limiting conditions. This assumption may be unconservative in view of Meyerhof's (1963) results and does require verification.

The compressibility factors assumed also require verification because of the uncertainty in the assumed soil shear modulus, G, for the sediments in question. Reliable estimates of the soil shear modulus, G, are required in order to select reliable compressibility factors. Those factors that were selected and listed in Table 17 are best estimates given the available information, but they do require in-depth review.

Results.

1. Cohesive. the bearing capacity analysis indicates that those deadweight anchors on cohesive materials will not fail in a bearing capacity mode. Figure 51 illustrates the relationship of bearing capacity, Q, to the lateral load capacity, R_L, as a funciton of anchor width, B, on a category A soil. the bearing capacity, Q, was calculated assuming the load's resultant applied to the soil was inclined at 0.79 rad (45 deg). The deadweight was also assumed loaded to ultimate laterally or P_H (load) = R_L (ultimate). Then given the resultant inclination of 0.79 rad, the vertical load component, R_V, for the analysis is equal to the lateral load capacity, R_L. Thus, the R_L curve in Figure 51 also represents the vertical load component R_V, applied in the anlysis. Obviously, the bearing capacity, Q, exceeds the applied vertical load component, R_V, by some 750 percent, thus failure in a bearing capacity mode is very unlikely.

Figure 51 also addresses the driving force necessary to embed the cutting edges, Q_e. Q_e is shown to be slightly greater than the vertical load component. $R_V = R_L$, assumed in the bearing capacity analysis. The deadweight anchor would have to weigh the greater amount, Q_e, in order to embed the cutting edges. This anlysis assumes a zero degree mooring line angle, thus no additional weight is required to balance a vertical component from the mooring line. The submerged weight required for other mooring line angles (assuming H/B = Z/B = 0.1) is given by:

$$R_R = R_L + R_L \tan \beta \text{ but not less than } Q_e \qquad (24)$$

where R_R = Required submerged weight (N)

β = Mooring line angle with the seafloor (rad)

Note that the bearing capacity plotted in Figure 51 was based on an assumed submerged weight equal to the applied lateral load. A more precise value of bearing capacity could now be calculated using the required submerged weight (R_R) as the vertical load. This was not done since the resulting change in bearing capacity is minimal.

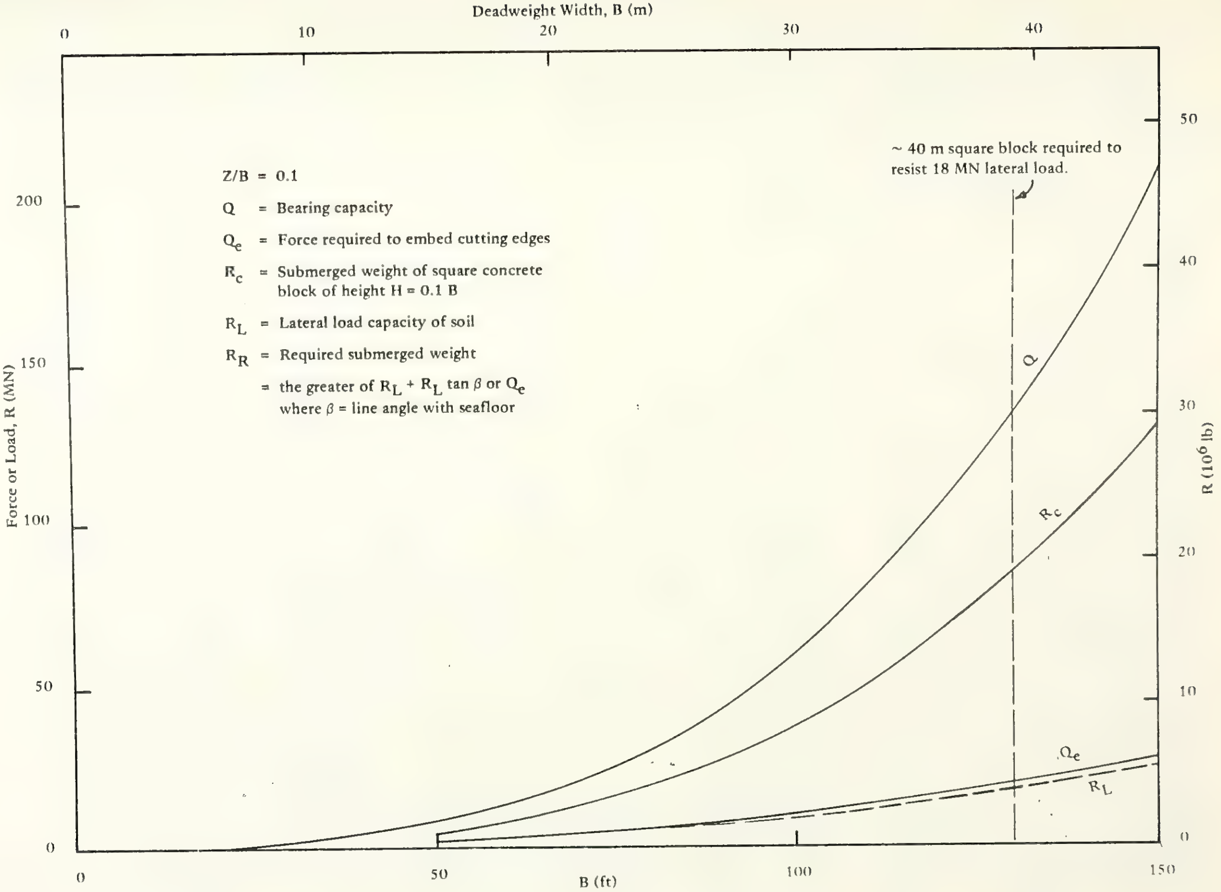


Figure 51. Relation of deadweight bearing capacity to various other loadings in soil category A.

The Q_e values shown in Figure 51 and elsewhere in this report were calculated assuming that the skirts penetrate evenly and vertically. A canted or inclined deadweight would produce uneven penetration and would result in higher embedment force. Thus, development of means to ensure uniform penetration of the large plan area deadweights required of OTEC may be necessary if uneven penetration is in fact a problem.

The R_C curve represents the submerged weight of a block of prestressed concrete of dimensions B by B square and 0.1B high, the dimensions assumed for the OTEC deadweight anchors. The potential submerged weight, R_C, of a block of these dimensions is 200 percent greater than the weight required to embed the cutting edges, Q_e. Thus, considerable "excess" vertical load carrying potential is available should a non-zero mooring line angle be necessary or should additional weight be required to account for uneven penetration.

On a category C soil (cohesive and strong) the relationship of bearing capacity to the loads being applied is much the same, that is, the applied vertical loads will be considerably lower than the soil bearing capacity (Figure 52). However, the relationship of the loads R_L, Q_e, and R_C shifts somewhat. The required submerged weight on the seafloor is again given by Equation 24. As before, the assumed volume of the anchor block, B by B by 0.1B high, provides ample space for weighting material with density equivalent to prestressed concrete.

2. Non-Cohesive. On non-cohesive sediments, Category D soil, the lateral load capacity is influenced much more drastically by load inclination as compared to the inclined load capacity of cohesive soils. Figure 50 shows that for a load inclination of 0.79 rad (45 deg), the inclination factors i_γ and i_q in the bearing capacity equation, (23), are equal to zero. Since the undrained shear strength, s, is also zero for non-cohesive soils, the bearing capacity is exceeded. Figure 50 illustrates that the vertical load component, R_V, must be increased for any given lateral load, P_H, in order to increase the magnitudes of the inclination factors i_γ and i_q. Figure 53 describes the interdependence of R_V, the applied vertical load, and Q, the resulting realizable bearing capacity. As the applied load R_V is increased, the bearing capacity, Q, is increased, but at a much greater rate. At some point the applied load and the bearing capacity will balance each other. For the case in question, balance occurs at a load component ratio, P_H/R_V, of 0.65 corresponding to an applied vertical load, R_V, of 280 MN (62×10^6 lbs) for an applied lateral load, P_H, of 180 MN (40×10^6 lbs).

The authors note that a bearing capacity type failure on sand may not be a serious problem. As discussed in the first report, the major result of a bearing capacity type failure is line abrasion on the high side of the block as the load direction shifts. In a uni-directional loading environment (Gulf Stream) this line abrasion should not occur. Model testing to be carried out in the next phase of the OTEC effort should clarify the seriousness of a bearing capacity type failure. The extremely large weights indicated by the analysis above may in fact be unnecessary if a bearing capacity type failure is found to be of minor consequence.

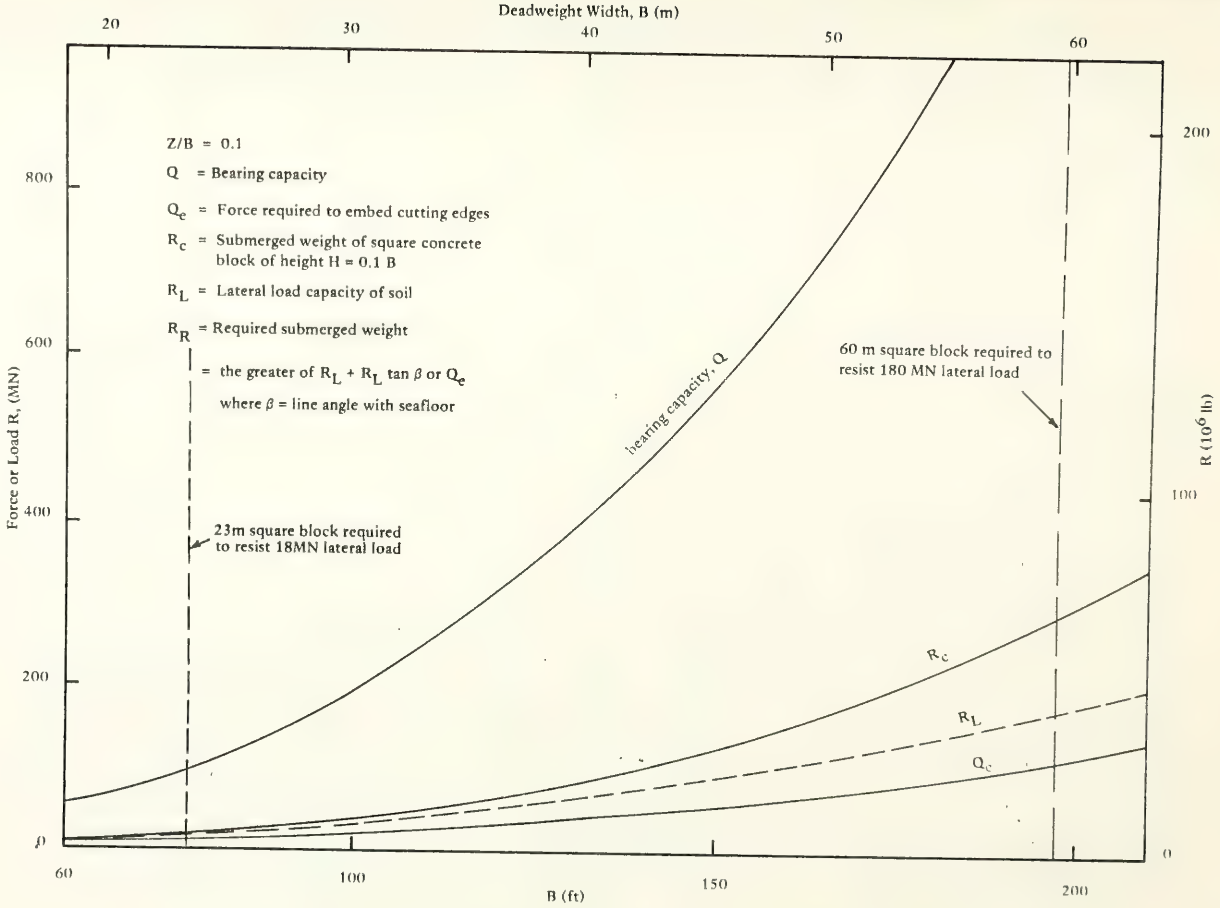


Figure 52. Relation of deadweight bearing capacity to various other loadings in soil category C.

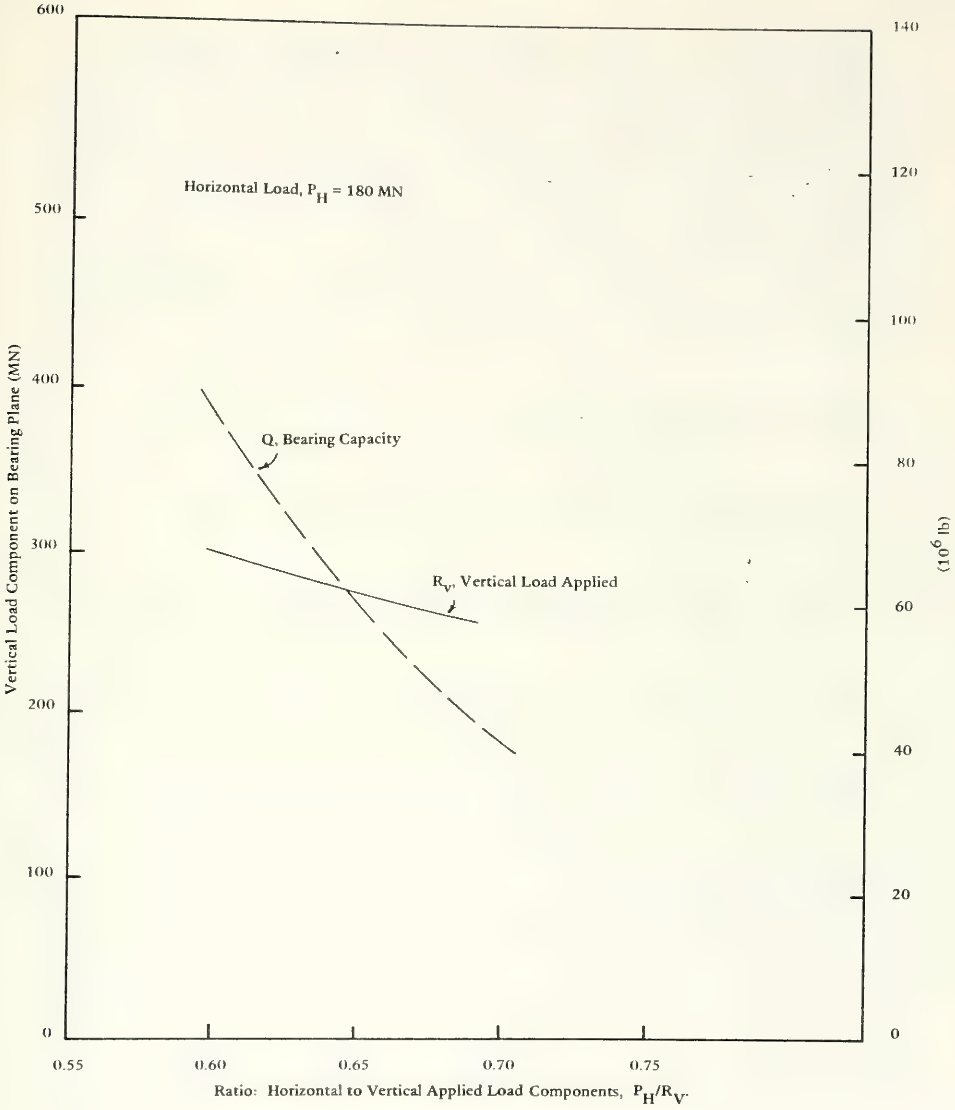


Figure 53. Influence of load inclination on bearing capacity, soil category D.

Figure 54 illustrates the relationship between the various loads involved with a deadweight anchor on sand as a function of the square block width. The bearing capacity, Q, is equal to the required vertical load ($Q = R_R \simeq 1.5\ R_L$). The force Q_e, required to embed the cutting edges, of length $Z = 0.05B$, is about 25 percent of the required vertical load, R_R, thus no embedment problems are foreseen. In order to achieve the submerged weight required, R_R, within the assumed anchor dimensions some special weighting may be necessary. This is so because the weight of a solid, prestressed concrete block, B by B by 0.1B, is less than the block weight, R_R, required to achieve a like bearing capacity. The difference in weight between R_R and R_c is relatively small and supplying that weighting, by increased bulk density, should present no problem.

Findings and Conclusions

1. The optimum ratio of cutting edge length to square deadweight side dimension, Z/B, varies primarily with the soil type:

    a. For cohesive soils, the optimum Z/B ratio is 0.1
    b. For non-cohesive soils, the optimum Z/B ratio is about 0.05 or somewhat less.

2. Deadweight anchor (square) dimensions necessary to maintain OTEC on station range between 22 and 60 m depending on the loading condition and seafloor material (see Table 15).

3. In order to prevent excessive tilting of a deadweight anchor (local bearing capacity failure), the effective weight (vertical force component R_v) applied to the seafloor must meet certain minimum values:

    a. For cohesive soils, the required weight is given by Equation 24. For the cohesive soils and loads considered, the required embedment force is approximately equal to the applied lateral load.

    b. For non-cohesive soils, the required weight is dictated by bearing capacity, not skirt embedment. To prevent a bearing capacity type failure, a submerged weight of approximately 1.5 times the applied lateral load is necessary. The tremendous weight magnitudes imposed by this requirement may be reduced if testing indicates that a failure in the bearing capacity mode will not lead to catastrophic anchor failure. Also, the coefficients used in the bearing capacity equations need verification for the OTEC loading conditions. Present uncertainties in these coefficients force conservative design. Model tests to reduce this problem, and hopefully to enable deadweight design to be guided by lateral resistance rather than bearing capacity in non-cohesive soils, are included in the next phase of the OTEC effort.

4. The concept of using a deadweight anchor with cutting edges to moor OTEC platforms on sediment seafloors is viable.

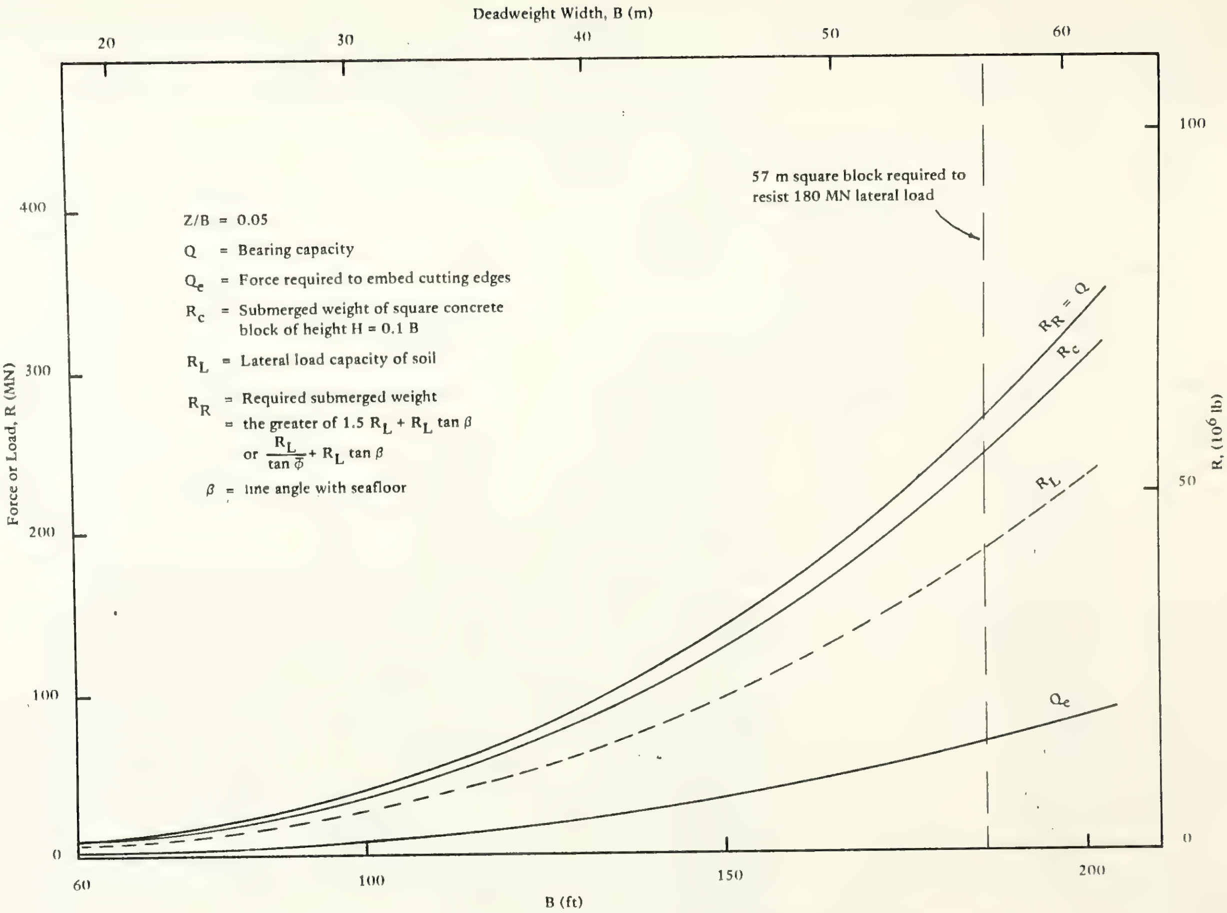


Figure 54. Relation of deadweight bearing capacity to various other loadings in soil category D.

GROUP CAPACITY

Factors Affecting Group Capacity

Introduction. Data available from which to determine group capacity is extremely limited; thus, maximum use of related data was made. The factors affecting group capacity are not well understood. Depending on the theory or test results one chooses, the following appear to have an effect:

1) Spacing
2) Relative embedment depth
3) Soil characteristics
4) Anchor type and characteristics
5) Anchor number and arrangement
6) Method of installation
7) Loading geometry (axial, lateral)

The only factor which stands out enough to be considered explicity is spacing. All theory, model results, and the limited field data available for soft seafloors indicate that decreasing anchor spacing decreases group capacity. Recommended spacings will be discussed and categorized according to anchor type and soil type. Suggested procedures must be considered interim due to the limited data; designs based upon these must use sufficient factors of safety. Once the primary OTEC anchoring choices are defined then the needed data will have to be gathered in order to justify reduced design safety factors.

Clay Seafloor. After placing anchor flukes, either through dragging, pile driving, augering, ballistic driving, etc., their performance as a function of spacing should be similar. An approach for determining desired spacing in clay was derived from data presented for the prediction of anchor holding capacity by Taylor and Lee, 1972. These data were replotted in Figure 55 as relative anchor spacing versus soil strengths. Minimum required embedment depth is also indicated by the abscissa. For example, if two anchors of width B in 28 kPa (4 psi) shear strength soil are to exhibit maximum efficiency then they must be placed at least 5B center to center.

As the anchors are placed closer together, the efficiency of the group is reduced because the individual anchors behave as a single unit. The failure mode change may be of three types. First, if embedded near the soil surface, the failure mode could change from the deep plate mode for a single embedded plate at Z; to the shallow plate mode for a group of closely spaced plates all at depth Z. A group of three or more plates, embedded to a soil depth near their individual critical embedment depths, would act as one large plate at the same depth Z and the shallow failure mode would occur (see Taylor and Lee, 1972). This failure mode change could reduce the average holding capacity of each anchor in the group to 40 percent of that of the original, uncombined, originally-spaced anchor capacity.

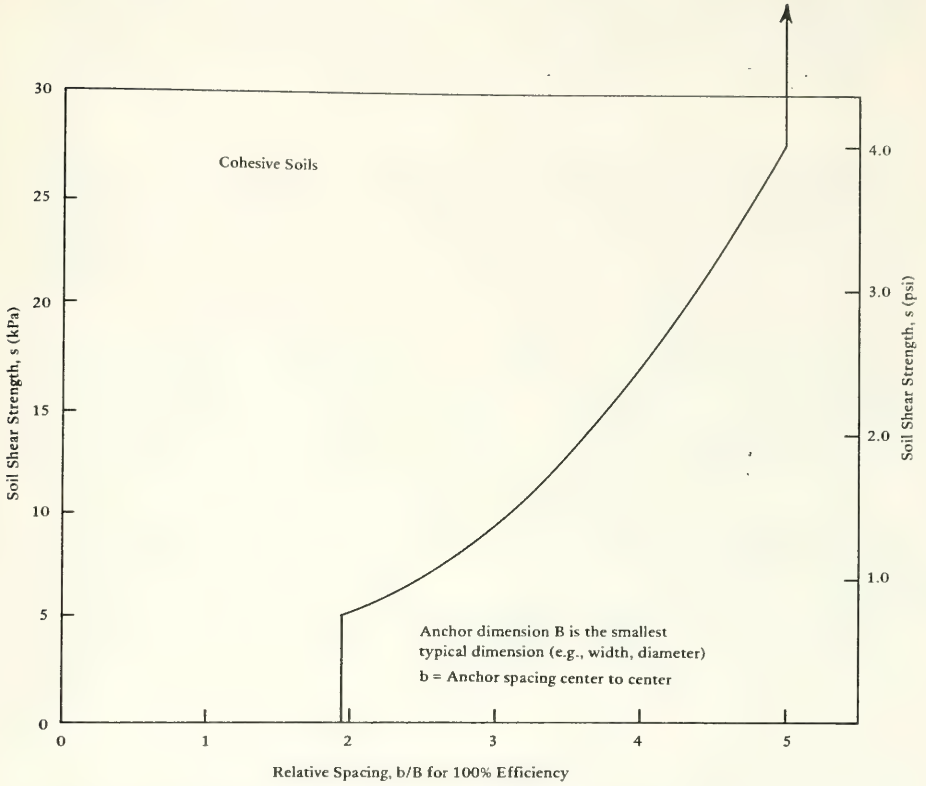


Figure 55. Minimum spacing of embedded anchor flukes for cohesive soils to realize 100 percent group efficiency.

Second, if the plates are quite deep but still at the same depth Z, then when brought close together, their failure zones would interact to reduce their capacity. For instance, if we were dealing with equal square plates and one edge of each were to touch to form a rectangle, then the group holding capacity would be reduced to 92 percent according to the shape factor.

$$S_p = 0.84 + 0.16\ B/L \qquad (25)$$

Third, if the plates were stacked one atop the other like the multiple helices of a screw anchor or stacked one behind the other like multiple drag anchors in series on one line, then close spacing of the flukes will result in shearing the soil on the surface of a cylinder or prism encompassing the flukes. This latter mechanism can drastically reduce group efficiency.

The relationship of Figure 55 might be questioned for individual anchors spaced perpendicular to the direction of pull, because the relationship is based primarily upon the effect of the individual anchors being spaced parallel to the direction of load. However, the results of Langley (1967) show that for groups of bulbous piles in soils of shear strength 24 to 124 kPa (3.5 to 18 psi), a spacing of 5 diameters was sufficient to develop the full pullout resistance of individual piles. These data agree well with the data of Figure 55.

Sand Seafloor. Minimum spacing for two anchors for 100 percent efficiency is plotted in Figure 56. This data was also taken from Taylor and Lee, 1972 and replotted for simplicity. Typical friction angles of a seafloor sand will range from 0.52 to 0.61 rad (30 to 35 deg); thus minimum required spacings will be 4 to 5 times anchor width or diameter. The same limitations regarding performance of anchors in clay spaced closer than their minimum holds for anchors in sand.

Anchor Spacing Recommendations

Conventional (Drag Burial) Anchor Spacing. Conventional anchors can be used singly, or multiply (in tandem or in parallel). The spacings for 100 percent efficiency in Figures 55 and 56 are recommended. When multiple anchors are used, their group capacity will depend not only upon their spacing and depth of embedment, but also upon how they are connected and installed.

To achieve proper spacing, anchor drag distance for an efficient burial anchor is about six times embedment depth (embedment depth estimated from Figure 5). For anchors attached in parallel (Figure 57a) the anchors must be spotted (before setting) a sufficient distance apart to insure that their final spacing (after setting) will be adequate. A somewhat different problem occurs with anchors placed in tandem (see Figure 57b). If the anchors are not efficient burial anchors, for example, Doris mud anchors (refer to section on conventional anchor extrapolation), then the minimum desired spacing should be increased to prevent the "following" anchors

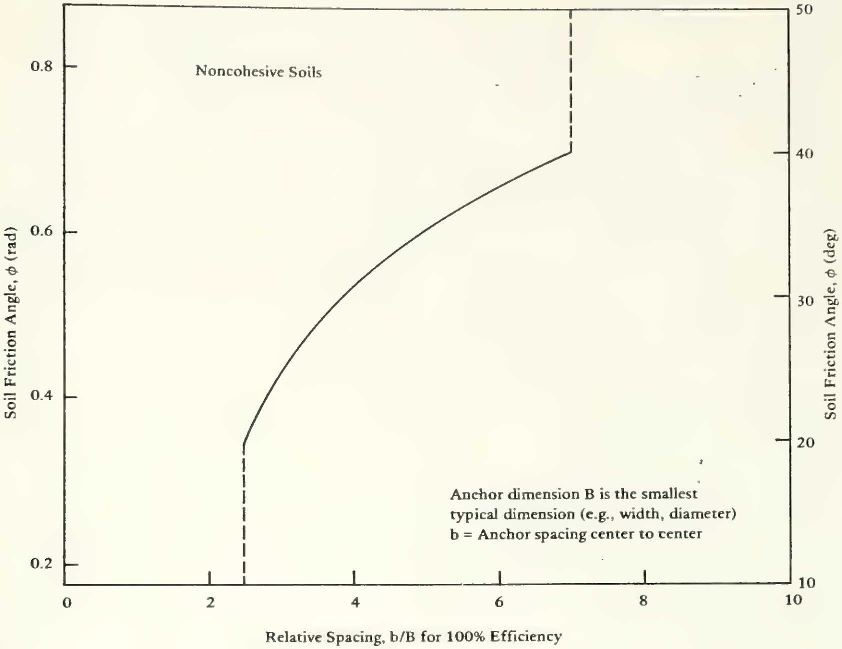
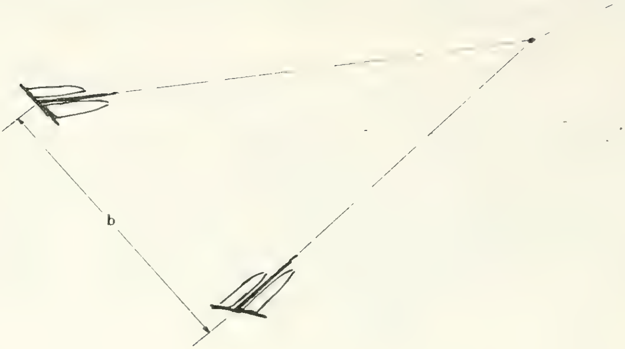
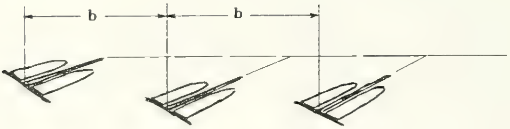


Figure 56. Minimum spacing of embedded anchor flukes for noncohesive soils to realize 100 percent group efficiency.



(a) Parallel.

b = anchor spacing for 100% efficiency



(b) Tandem.

Figure 57. Tandem and parallel anchor configuration.

from penetrating the soil disturbed by "prior anchors. The disturbed area could cause a considerable reduction in holding capacity in a sensitive seafloor soil.

Plate Anchor Spacing. Plate anchors should be spaced according to the relationships provided in Figures 55 and 56.

Pile Anchor Spacing. A large amount of data is available on the behavior of single piles. The difficulty in applying required test loadings has resulted in little performance data for full scale pile groups. This lack of group pile data has resulted in attempts to use single pile data to predict group capacity. One approach has been to define a group efficiency equal to the actual group capacity divided by the sum of individual pile capacities. A typical efficiency formula is shown below. This is known as the Converse-Labarre equation.

$$\eta = 1.00 - 57.3 \frac{D}{b} \times \frac{(n-1)(m) + (m-1)(n)}{90mn} \tag{26}$$

where η = group efficiency

D = pile diameter (m)

b = pile spacing (m)

m = number of rows

n = number of columns

The equation is based on overlapping of bulb stesses around piles which causes a reduction of soil capacity. The Converse-Labarre equation may be used for approximate prediction of friction pile group pullout capacity.

Rough estimates of lateral load group efficiency may be obtained from model tests (Poulous, 1975). No actual field data is available for laterally loaded pile groups in soft seafloor-type materials.

Practical considerations concerning spacing of driven piles were stated by Terzaghi and Peck (1967); their observations appear applicable to uplift or lateral resisting pile groups. They recommend that the distance between pile centers should not be less than 2½ diameters. If spacing is less then the heave of the soil is likely to be excessive and the driving of each new pile may displace or lift the adjacent piles. On the other hand, a spacing of greater than four diameters is uneconomical because it increases the cost of the pile cap without materially benefitting the foundation/anchor.

For intermediate and final design of groups in particular soils a detailed analysis using one of the rational techniques available (Terzaghi and Peck, 1967; Focht and Koch, 1973; Murthy and Shrivastava, 1972) is recommended.

ANCHORS ON ROCK

Introduction

The special anchoring requirements of rock seafloors are discussed in this section. Rock will probably be encountered for some OTEC sites in the Gulf Stream. Thus, the difficulties of anchoring in rock are compounded with the problems imposed by large mooring loads.

There is relatively little experience in rock anchorages, and certainly no experience with the load magnitudes possible with OTEC. Consequently, the anlysis below is conceptual. The designs cited are basic ideas which will be refined, altered or combined to achieve optimum performance for a particular situation.

Environment

A typical site in the Gulf Stream would have a water depth of about 460 meters. It would have a karst-like, possibly cavernous limestone topography. The University of Miami has made several dredge hauls in such areas on the Miami Terrace. Their samples were identified (X-ray crystallography) as amorphous flour-apatite; a rock in which the carbonate has been replaced by phosphate and flourine ions. The vertical extent of the rock is unknown. Samples taken were finely laminated. They were composed of alternating fossiliferous and barren layers. Foraminifera were predominant, but not excessive. Deposition was probably Tertiary.

Very little is known about the engineering properties of Gulf Stream rock. This is unfortunate since knowledge of rock strength characteristics is of primary importance in anchor design. Obtaining representative data is a difficult and an expensive task. The engineering properties of marine limestone may vary drastically within a small area (Yang, 1976). For this reason a broad general sampling program would be of little value. Sampling and testing should be done carefully at the actual site location. Actual site topography and rock properties are essential for an optimum anchor design.

Basic Anchor Concepts in Rock

The several basic anchor concepts described below are summarized in Table 18. All anchors were designed assuming that horizontal and vertical force components of 180 MN (40×10^6 lbs) were applied at the anchor.

The seafloor was assumed to be composed of uniform limestone with an average compressive strength of 29 MPa (4200 psi) (Farmer, 1968). A coefficient of friction between concrete and rock of 0.3 was assumed for calculations involving anchor sliding. The bond strength in shear between concrete and a natural rock surface was assumed as 69kPa (10 psi). A bond strength in shear of 517 kPa (75 psi) between grout and steel and between grout and rock was used for piles and tendons in drilled holes.

Table 18. Anchor Concepts for OTEC in Gulf Stream,[a] Rock Seafloor

Description	Dimensions (m)	Weight[b] (MN/10^6 lb)	Assumptions	Advantages	Disadvantages
Simple deadweight	83 x 83 x 8	780/175	Sliding failure[c]	1. Simple, reliable system	1. Large weight 2. Bulky 3. Difficult to transport and lower to seafloor
Grouted deadweight	51 x 51 x 6	180/40	Sliding failure[d]	1. Reduced size	1. Requires site preparation 2. Requires high grout bond quality 3. Difficult to transport and lower to seafloor 4. Requires near-flat, rock seafloor
Deadweight in a sinkhole	57 x 57 x 6	255/57	No lateral sliding	1. Reduced size 2. Simple installation	1. Application limited 2. Line abrasion likely
Piles (drilled and grouted) Requires 144 0.25-m diam 20 m long steel piles	20 x 20 x 1	13/2.9	Rock crushing[e] or block uplift[f] failure	1. Reduced size and weight	1. Complex installation, requires development of seafloor drilling machine 2. Requires increased on-site construction time 3. High cost
Pre-stressed steel wire tendons (drilled, tensioned, grouted) Requires 169 0.06-m diam equivalent, 20 m long steel tendons in drilled holes (0.12 m x 20 m)	20 x 20 x 1	11/2.5	Sliding or block uplift failure	1. Reduced size and weight	1. Complex installation, requires development of seafloor drilling machine 2. Requires increased on-site construction time 3. Requires development of seafloor tendon tensioning equipment 4. High cost
Plate anchor in sinkhole	13 x 13 x 0.7	8/1.8	Fill material and suitable sinkhole available	1. Simple installation 2. Reduced size and weight	1. Limited application 2. Line abrasion likely

[a] Horizontal and vertical load components = 180 MN (40 x 10^6 lb).

[b] Submerged densities: concrete = 13.5 kN/m^3 (86 pcf), steel = 67 kN/m^3 (426 pcf), rock = 11 kN/m^3 (71 pcf).

[c] Coefficient of friction concrete-rock = 0.3.

[d] Bond strength grout-rock = 69 kPa (10 psi).

[e] Compressive strength of rock = 19 MPa (4200 psi).

[f] Coefficient of friction along vertical rock fracture = 0.5.

Several reported designs use similar values (Sakuta et al., 1975; Mutoh, 1975).

Simple Deadweight. As shown by Table 18, a deadweight of 780 MN (175×10^6 lbs) is required to resist Gulf Stream loading. The extreme size (83 m x 83 m x 8.3 m) of this anchor make handling, transportation and installation major problems.

Two possible techniques for accomplishing these tasks are suggested.

First, the anchor could be fabricated in a large dry dock, then barged or towed to the site using external or internal buoyancy tanks. A buoyancy or heavy lift system would also be required to lower the anchor to the seafloor. Second, the anchor could be cast in place on the seafloor. Again, a heavy lift sytem would be needed to pre-position the framework on the ocean floor. Technology for pouring concrete in the ocean at a depth of 460 m has not been demonstrated to date, but is probably within reach in the near future.

The advantages of the deadweight are its simplicity and reliability. It does not require that complex underwater operations be carried out in the high energy Gulf Stream environment; and it is not likely to fail catastrophically.

Note that for all deadweight calculations a height to width ratio of 0.1 was used. A ratio of as much as 0.4 is probably acceptable on a rock seafloor. This would reduce the lateral dimension shown in Table 18 to about 0.7 of the stated values.

Grouted Deadweight. A substantial reduction in anchor size results if grout, injected beneath a deadweight, is used to resist lateral load. Since grout is ineffective in tension, the deadweight described in Table 18 was designed with a mass sufficient to resist the vertical load. The shear strength of the grout-rock bond is assumed to resist lateral load. A bond shear strength of 69 kPa (10 psi) is required over the entire deadweight bottom surface to resist the 180 MN lateral load. Careful seafloor surface preparation (possibly by suction dredging) is required to insure sufficient bond integrity.

Again, a heavy lift system would be needed to install the anchor. By grouting or gluing the dead weight to the underlying rock, size is reduced, but cost is increased, installation complexity is increased, and a questionable bond with the seafloor exists.

The grouted deadweight concept might be carried one step further. Use of an underwater curing epoxy, instead of a coment grout, could reduce anchor size drastically. Epoxy grouts can sustain tension as well as shear. However, there would be an even more critical dependence on the bond at the seafloor. Experience with underwater epoxies is limited. Their long term performance in a marine environment under heavy loads has not been proven conclusively. Further, bonding of the anchor to a cleaned rock surface may not increase anchor holding capacity since the rock itself may separate along bedding planes. Thus, bonding of an anchor block to a rock surface would be an uncertain operation at best.

Deadweight - Sinkhole Combination. Sinkholes large enough to be explored by submersible occur frequently on the terrace off southeastern Florida. Placing a deadweight into a singkhole would alter the mechanism for resisting the lateral load component. Much or all of the lateral load component would be transferred directly from the mooring line to the rock at the lip of the sinkhole. The required deadweight size could be decreased accordingly. For example, the anchor in Table 18 was designed to resist a maximum line tension of 180 MN x 1.41 = 255 MN.

Two major difficulties with utilizing this concept are: 1) there is no guarantee that a suitable sinkhole will be near an acceptable OTEC site, and, 2) line abrasion at the lip of the sinkhole would be severe.

Driven or Drilled Piles. The behavior of piles in rock is not clearly defined. Information dealing with pile systems in rock in the marine environment is limited. The amount of practical installation experience is likewise limited. However, pile systems remain a strong contender for the OTEC anchor in rock because: (1) pile systems are much less dependent on seafloor surface irregularities than a simple or grouted deadweight and (2) piles also substantially reduce the required dimension and material weights as shown in Table 18.

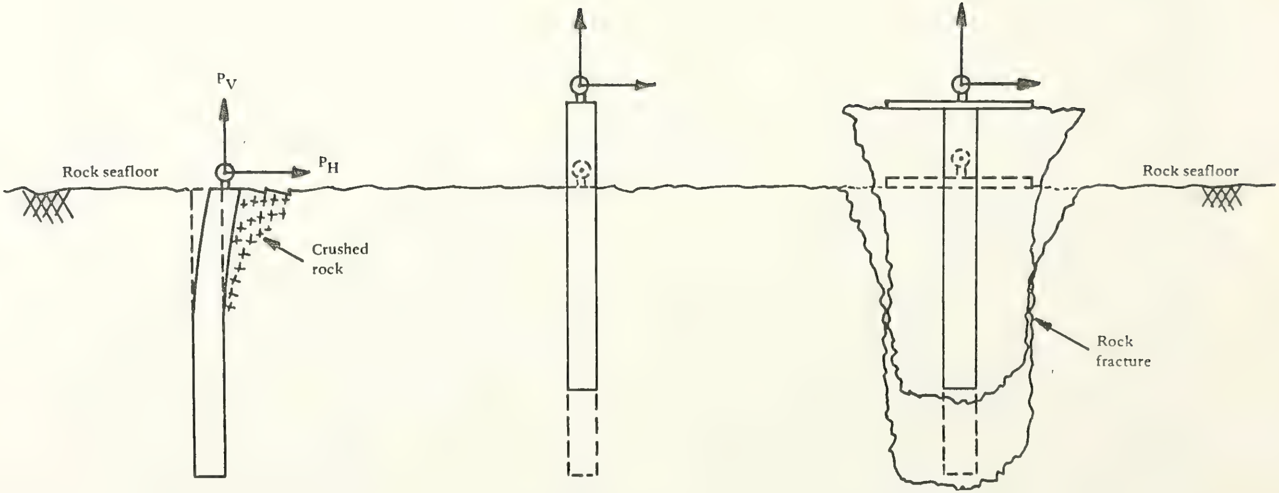
An OTEC anchor cmprised of a group of bored and grouted piles in rock could potentially fail in three simple mechanisms: (see Figure 58)

1. The individual piles will exert high bearing loads against the restraining near-surface rock and could cause crushing of that rock. Crushing of the near-surface rock would allow pile lateral deflection with accompanying increase in bending moment in the pile, and a shifting of restraining load to lower rock layers. The piles could fail in bending or the crushing of rock could progress downward until the piles pulled out.

2. Axial load in a pile could exceed the capacity of the grout to steel bond, the grout to rock bond, or the grout shear strength. Thus, excessive axial load could cause axial pullout of a grouted pile.

3. The rock mass into which the piles are grouted may be weak or fractured. Uplift failure of the entire rock mass could occur. If we assume a weathered, layered limestone, as appears from the limited dredge hauls, along with some jointing, this assumption is likely the case. To reduce the likelihood of such failure, the site survey must identify the probable rock strength and joint and fault systems, and the pile system must be designed to entrain a sufficient mass of rock to provide the necessary vertical restraint.

The OTEC anchor could also fail in combinations of the above mechanisms depending primarily on the system of discontinuities in the rock. The following analysis considers only the above three simplistic mechanisms.



(a) Lateral failure of rock.

(b) Uplift failure of grout bond.

(c) Uplift block failure of rock.

Figure 58. Assumed failure modes for pile anchors on a rock seafloor.

First, the assumption was made that the entire lateral load on a pile is absorbed by the first 0.3 m of rock adjacent to the pile below the seafloor. Using a conservative limestone compressive strength of 29 MPa (4200 psi), and assuming a 0.25 m diameter pile, lateral resistance per pile was found to be 1.43 MN (320,000 lbs). Roughly 125 such piles are required to resist a lateral load of 180 MN. For simplicity a 12 by 12 pile array (144 piles) was assumed.

Next, the adequacy of the group design against vertical pullout was treated. Failure mode three (Figure 58c) was found to be more stringent than mode two (Figure 58b) for the pile spacing and rock condition assumed. Therefore, the pile group in Table 18 was designed to resist uplift failure of the entire block. In other words, pile spacing and length were designed so that the weight of entrained limestone (11 kn/m^3) plus friction developed along the vertical sliding surface balanced uplift force.

Analysis of other cases in the literature suggests that the procedure used is conservative. Even with this conservativeness, a significant anchor size and weight reduction, as compared to a pure deadweight was indicated.

The cost of drilling and grouting the required 144 piles would be high. However, by increasing pile size or perhaps by using a deadweight and battered pile combination, this cost might be reduced. Selection of an optimum design requires a refined analysis procedure based on actual rock properties and loading conditions.

Pre-stressed Tendons. A similar reduction in anchor size might be achieved by using pre-stressed tendons. First, holes would be drilled through a template into the rock seafloor. High tensile strength steel tendons would then be inserted and grouted in place. Finally, each tendon would be tensioned by jacking down against a template on the seafloor. The template would be forced against the seafloor with a normal force large enough to provide the required frictional resistance to sliding. In effect, the tensionong force acts as an equivalent deadweight force.

For example, using the same coefficient of friction as for the simple deadweight, a tensioning force of 780 MN is required. The total cross-sectional area of steel tendons required is 0.5m^2 assuming 250 ksi (1725 MPa) steel. As was the case for piles, the uplift failure mode shown by Figure 58c governs. Therefore, a template of the same size (20 m), and steel tendons of the same length (20 m as for piles is needed to entrain the required limestone mass.

Examination of Table 18 shows that little is gained by using post-tensioned tendons rather than piles. The intricacy of installation and resulting high cost would reinforce this conclusion.

Plate Anchor-Sinkhole Combination. Another anchor concept which was considered again asssumed the availability of a suitable sinkhole near the OTEC site. A large plate is placed into the sinkhole and covered with a fill material. This would remove the major problem of embedment of the large plate size required for OTEC. Approximate plate size was calculated as before and is repeated in Table 18.

This concept faces the same difficulties as noted for a deadweight in a sinkhole, notably limited applicability and line abrasion. In addition, acquiring the necessary fill material would be a formidable task.

Conclusions and Recommendations

Anchor design for a rock seafloor site in the Gulf Stream requires specific knowledge of the local topography as well as rock engineering properties. The investigation revealed that anchoring in such a location will present problems not encountered on normal seafloor sediments. A commensurate increase in anchor cost is probable.

Use of the pile anchor system for the Gulf Stream rock environment is very attractive at present because the pile anchor system can easily accomodate much of the topographic irregularity found in the rock environment and because pile configuration can be altered on site to accomodate variable rock properties. However, pile installation in 460 m of water on an exposed or thinly covered rock seafloor will require the development of some new technology. To resist uplift loads, piles in the rock environment will probably be grouted into bored holes. The equipment for starting such bored holes in thinly covered rock from a floating platform is not available. However, the necessary equipment is not overly complex when compared to present offshore oil-production technology.

Deadweight anchors (simple or grouted) would be an excellent choice for OTEC if: 1) a heavy lift capability is available for placing the anchor and, 2) the ability to pour large concrete masses on the seafloor is demonstrated. Simplicity, economy, and reliability are the primary advantages of deadweight anchors.

SUMMARY

Deadweight Anchors

Deadweight anchors have been found very suitable for mooring the OTEC platform, especially on unconsolidated sediments. On unconsolidated sediments, the deadweight is fitted with cutting edges about its periphery and at intermediate locations beneath the anchor block. The anchor weight in cohesive soils is dictated by (1) the force required to embed the cutting edges or (2) to prevent overturning, whichever is larger. The cutting edges serve to depress the failure plane in lateral sliding into deeper and stronger soil strata. Through this mechanism, the lateral holding capacity to weight ratio of the deadweight (assumed concrete) is about 1/2 to 1 (1 to 1 in terms of submerged weight) on cohesive soils. On non-cohesive soils (sands), the lateral holding capacity to weight ratio is about 1/3 to 1 (2/3 to 1 in terms of submerged weight). In this latter case, on sand, the mass required for the deadweight may not be dictated by the weight required to prevent sliding along the base shear plane. Rather, the mass may be dictated by a bearing capacity problem beneath the leading edge of the anchor block: on sands the anchor block is in danger of failing locally in bearing capacity and nosing over into the sediment.

To counter this tendency, the net downward force on the sediment must be increased to increase the effective normal stress across the potential shear zone. Raising the normal stress raises the shear strength of the sand and increases the bearing capacity. In other words, the weight of the block must be increased to prevent a bearing capacity type failure. This means a large increase in the material and installation cost of the anchor. Further work to be done in the next phase of the CEL OTEC effort will determine the consequences of such a failure. Model tests and refinements of the analysis procedure could indicate that a bearing cpacity type failure would not adversely affect anchor performance in a unidirectional loading environment (Gulf Stream). Such a result could mean substantial savings in hardware and installation costs.

The use of deadweights on exposed or shallowly buried rock surfaces, such as found on the floor of the Gulf Stream, offers some special problems. The rock surface is relatively unyielding so it is difficult for cutting edges to bite into it. Nor will the rock surface conform to the deadweight bearing surface. As a result, it is difficult to achieve sufficient lateral load resistance with a deadweight. Effective coefficients of friction on rock are about 0.3, leading to a lateral holding capacity to weight ratio for a deadweight of 1/6 to 1 (1/3 to 1 in terms of submerged weight). On rock, it appears prudent to provide additional techniques for developing lateral load resistance. Short stub piles acting in shear appear excellent candidates for this purpose. Installation of the piles would require some means of developing sufficient thrust on the drill bit in order to start the drill hole, but otherwise should be state-of-the-art.

Pile Anchors

Pile anchors are very efficient carriers of axial load. Thus, those piles installed vertically as from a drillship can economically resist very high vertical components of mooring line force. However, piles are not efficient in resisting large lateral loads in the almost normally consolidated soil profiles of the deep ocean. Soft surface sediments are not able to provide sufficient lateral load resistance to the deflecting pile. Thus, the load is transferred to lower elevations in the pile, and bending moments are increased. These bending moments dictate structural design of the pile. The weight of hollow cylindrical steel piles in about 20 percent of the lateral force being resisted (lateral holding capacity to weight ratio of 5 to 1). The steel pile sections must be pressure grouted in the bored holes, especially in the predominantly calcareous ooze sediments. Given the sophisticated construction techniques required, and given the resulting lateral load to weight ratio, pile anchors in typical unconsolidated seafloor sediments appear of marginal value to the OTEC mooring program. This conclusion presumes that low mooring line angles will prevail for OTEC mooring systems.

Conversely, piles would serve well as OTEC anchors if high mooring line angles were adopted for the OTEC system. There are several reasons why high mooring line angles should be avoided. First, high mooring line angles mean increased mooring line forces: for a mooring line angle of 0.79 rad (45 deg) the line tension would be 1.4 times the tension for a line angle of 0.0 rad; for 1.0 rad (60 deg) the line tension would be twice as large. Note that there is not a mooring line available to hold OTEC even at a zero line angle. Raising the line angle increases line tension and compounds the mooring line problem. High line angles also stiffen the mooring which increases dynamic loading at the anchor. In moorings with low line angles, most of the dynamic loading is damped out before reaching the anchor by line motion in the water. Preliminary tests at University of California, Berkeley, suggest that dynamic loadings could result in significant strength reductions in the calcareous ooze sediments that predominate in proposed OTEC siting areas. Thus, dynamic loadings on anchors on calcareous ooze should be minimized to prevent soil strength reductions and anchor pullout. High mooring line angles and resulting high dynamic stresses at the anchor should be avoided.

Pile anchors may serve well to moor OTEC to a rock surface such as exists beneath some parts of the Gulf Stream. The pile sections designed are really steel shear pins which are grouted into rock to provide additional resistance against uplift.

Plate Anchors

Plate anchors (direct embedment anchors, screw anchors, etc.) are not suitable for providing the holding capacity required for OTEC in either loading environment. For example, in the deep ocean environment, (assuming a zero degree line angle at the seafloor) restraint of OTEC would require a square plate 6.1 m (20 ft) on a side embedded to a soil depth of 30 m (100 ft). To key a 6 m direct embedment fluke approximately 12 m of travel would be necessary. Thus, the initial penetration required of the fluke would be 30 m + 12 m = 42 m (140 ft). A propellant system for embedding these flukes is considered not feasible. Other driving systems, while feasible, would require a complex and very large driving guide on the seafloor and would require considerable equipment development to guarantee the project.

Please note that the above deep ocean loading assumes a horizontal loading. Increasing the mooring line angle would increase the number of 6 m plates required, and would necessitate bridling to equalize the loading. CEL has performed this task in shallow water where assembly could be accomplished subaerially; but installing and bridling flukes remotely, in deep water would be extremely complex and expensive.

Standard Burial Anchors

Drag embedment anchors could be fabricated to provide the required holding capacity for OTEC in the benign, deep ocean environment.

An anchor weighing 64 Mg (140,000 lbs), providing a predicted lateral holding capacity of 18 MN ($4x10^6$ lbs), is feasible. However, drag embedment anchors have two strong disadvantages which render them undesirable for use in the OTEC mooring:

1. Drag embedment anchors must be loaded in only one direction, whereas loading in the deep ocean environment will not be so limited. The direction of load application can be limited by using a multi-point moor with a drag embedment anchor at each leg. This approach would, however, multiply the material cost of the moor by at least four.

2. Drag embedment anchors must be embedded with a zero mooring line angle and, for efficient operation, that line angle must remain near zero. In order to attain a near-zero mooring line angle, a very long scope of line must be used or "sinkers" in the form of concrete blocks of heavy chain must be attached to or integrated into the mooring line to absorb the vertical component of mooring line tension. Thus, the drag anchor is best used in combination with a deadweight: the deadweight resists the vertical component of load and the drag embedment anchor resits the lateral component of load. Installation of a single leg for a multi-point moor would require precise handling of a heavy deadweight and a large drag anchor simultaneously. This would be an intricate operation in deep water. Other anchor types offer greater versatility and savings in materials. Drag embedment anchors are not desirable for mooring OTEC in the benign deep ocean environment. Drag embedment anchors are even less suitable for use inmaintaining the OTEC plant on station in the Gulf Stream environment. To resist possible 180 MN ($40x10^6$ lbs) loads, eight to ten 64 Mg (140,000 lbs) drag embedment anchors, bridled together so as to equally distribute the load, would be required. Proper installation of this drag anchor group, and development and maintenance of the equally distributed loadings; would be very difficult, rendering the concept not suitable for use in the Gulf Stream environment.

CLOSURE

As a result of this effort, certain constraints on the OTEC mooring design, and indeed possibly on the OTEC plant design, are evident. These constraints have been alluded to here. Coherent presentation of these constraints is made in the final report on the Phase I OTEC Anchor System effort.

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SYMBOLS

A	Bearing area of deadweight anchor (m^2) or cutting edge tips
A_s	Side area (m^2)
B	Width of deadweight anchor block (m).
b	Spacing between cutting edges beneath deadweight, or anchor spacing (m).
D	Diameter of pile or circular anchor block (m).
d	Depth below seafloor surface (m).
d_c, d_γ, d_q	Factors to account for the shearing resistance of soil above the bearing plane of the anchor.
E	Young's modulus (Pa).
e	Distance of resultant force from centroid of base (m).
H	Height of deadweight anchor excluding cutting edges (m).
I	Polar moment of inertial of the pile (m^4).
i_c, i_γ, i_q	Factors to account for inclination of the resultant load.
K	Modulus of horizontal subgrade reaction (Pa).
K_p	Coefficient of passive lateral earth pressure.
K_s	Coefficient of lateral pressure on pile wall.
k_c, k_γ, k_q	Factors to account for compressibility of the support soil.
L	Horizontal length, as the long dimension of a rectangular anchor block (m).
M	Moment (Nm).
M_{max}	Maximum moment (Nm).
M_o	Moment at surface (Nm).
M_p^1	Maximum bending moment in a cutting edge per unit width of deadweight anchor (N·m/m).

M_x	Moment at a point (Nm).
N_c, N_γ, N_q	Bearing capacity factors.
n_h	Coefficient of horizontal subgrade modulus (Pa/m).
P	Force applied to an anchor by the mooring line (N).
P_H	Horizontal component of mooring line force at the anchor (N).
P_V	Vertical component of mooring line force at the anchor (N).
p	Soil pressure (Pa).
Q	Vertical load reaction supplied by soil (N).
Q_e	Force required to embed cutting edges (N).
Q_s	Total side friction on pile (N).
R	Anchor capacity (N).
R_A	Axial force capacity of pile (N).
R_b	Lateral load resistance developed below the cutting edges of the deadweight, base shear resistance (N).
R_C	Submerged weight of concrete deadweight anchor (N).
R_L	Total lateral resistance or capacity (N).
R_p	Lateral load resistance of passive wedge against leading cutting edge of deadweight (N).
R_p^1	Lateral load resistance per unit width of passive wedge (N/m).
R_R	Required submerged weight (N).
R_V	Net vertical force (N).
Sp	Bearing capacity shape factor.
S_t	Sensitivity = (undisturbed undrained strength) ÷ (remolded undrained strength).
s	Undrained shear strength (Pa).
s_c, s_γ, s_q	Factors to account for base shape.

s_f	Friction against cutting edge lateral area (Pa).
sm	Average undrained shear strength (Pa).
T	Relative stiffness (m).
t	Thickness of cutting edge (M).
t_w	Thickness of pile wall (m).
V	Shear force (N).
W	Weight (N).
W_b	Submerged weight (N).
W_{eff}	The effective weight or force on the base shear plane (N).
W_s	Submerged weight of soil entrained within the cutting edges and above the base shear plane (N).
x	Distance along pile below point of lateral load application (m).
y	Lateral deflection (m).
Z	Length of structural element (m).
β	Angle of mooring line with horizontal (rad).
γ	Buoyant unit weight (N/m^3).
δ	Effective friction angle of soil on pile wall or cutting edge wall (rad).
λ	Friction capacity coefficient.
$\overline{\sigma}$	Vertical effective stress (Pa).
$\overline{\sigma}_m$	Average vertical effective stress over pile length (Pa).
$\overline{\phi}$	Effective angle of internal friction (rad).

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